BUILDING CONSTRUCTION

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PROF. HENRY ADAMS, M.I.C.E.

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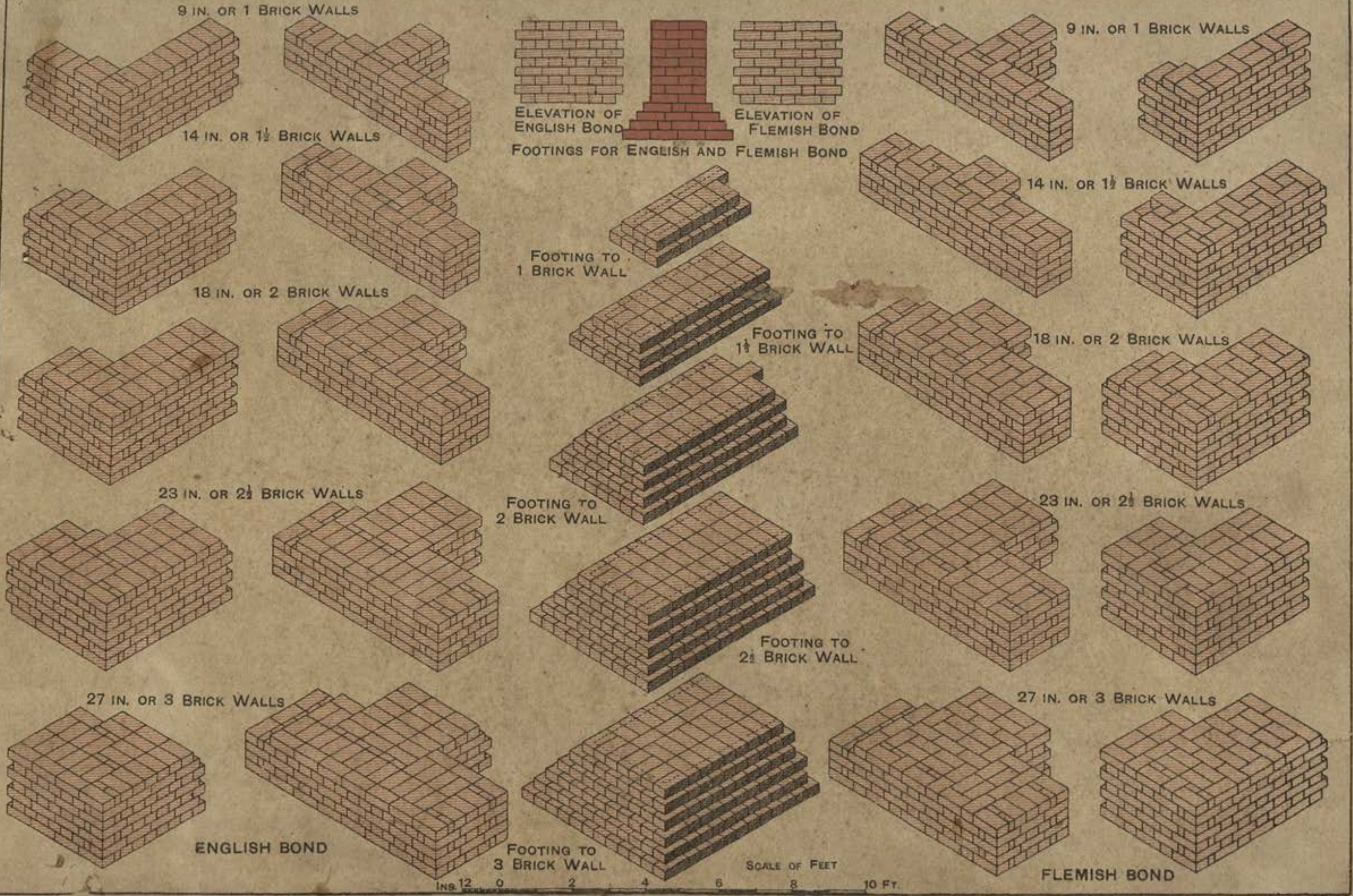
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FLEMISH BOND

BRICKWORK.

CASSELL'S

NG CONSTRUC

ING NOTES ON MATERIALS, PROCESSES,
PRINCIPLES, AND PRACTICE

BY

PROF. HENRY ADAMS, M.Inst.C.E., etc.

Examiner to the Board of Education, the Society of Architects,
the Royal Sanitary Institute, and the Institute of Sanitary Engineers

WITH 2,284 ILLUSTRATIONS AND 12 PLATES

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PUBLISHERS' NOTE

ASSELL'S BUILDING CONSTRUCTION is the most comprehensive, the most practical nd altogether the most valuable treatise on Building Construction ever issued. e name of the author is a guarantee of competency and thoroughness, Professor ams's extensive practice as consulting engineer, and architect and surveyor, aving afforded him extraordinary facilities for acquiring special as well as omplete general information on the subjects comprehended within the work, hile his distinguished reputation as a technological professor, and his position as aminer in Building Construction, etc., to the Board of Education, the Society f Architects, the Royal Sanitary Institute, the Institute of Sanitary Engineers, etc., ualify him to treat with unrivalled authority, as well as with exceptional clearness, e subject of Building Construction in all its aspects.

Students preparing for examinations in which Building Construction nvolved will welcome this as a text-book of the utmost value and importan nd its intensely practical character—in every possible instance the informatio imparted by investigating, describing, and illustrating cases that have occurr ctual experience—renders the work extremely useful as a guide to the everyd ractice of architects, builders, and craftsmen of the building trade, who will it virtually everything that relates to the materials, processes, princip ice of their respective callings.

he comprehensive scope of the work is evident from a glance at the list o ents. Each of the various sections is dealt with in exhaustive detail, some of sections extending to nearly 100 pages; and the studiously plain language d throughout the book is further assisted by the use of 2,284 skilfully drawn grams, which are supplemented by twelve full-page plates.

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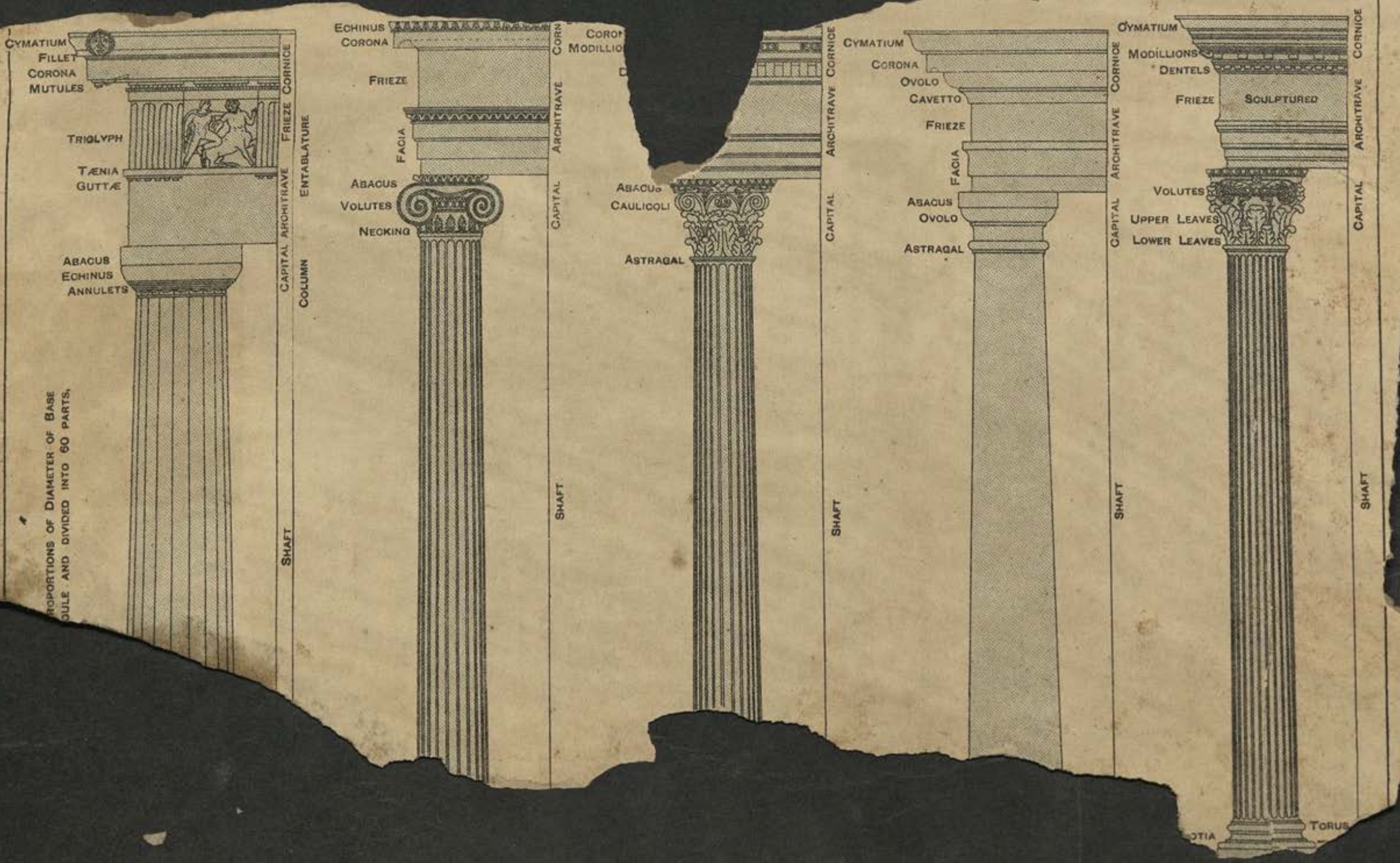
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BUILDING CONSTRUCTION

INTRODUCTION

Building Construction an Interesting Subject.

Building Construction is perhaps the subject of all others that possesses the widest interest among practical men, and it is not without considerable attraction to a large number of other persons of mature age, but to the student looking for a subject to specialise in it has charms that are not to be outweighed. Those who have a taste for drawing find new developments of their hobby, and follow it with tenfold interest because of its practical utility; while those who have already the practical faculty fairly developed seize upon the subject with avidity, since it shows them "the inside of things," and teaches them "how to make things." If there is such a phenomenon as a boy who did not pull his toys to pieces to see how they were made, his progress in the study of building construction might be slow; but most lads are so constituted that the more they learn about it the more they want to know, and "the boy is father to the man." Building construction, in short, is a subject of universal interest, and is, moreover, a most valuable instrument of practical education.

Practical Education.

The one great feature of our schools at the present day is the endeavour to make the education more practical, and in many schools "manual training" forms part of the ordinary course. This manual training is mostly confined to the use of joiners' tools, on account of the facility with which they can be used, the variety of work they will turn out, the use that can be made of the objects, the cleanliness of the operations, and the educational value of the work in the combined training of eye and hand. Other building trades might be taken up in a similar way, but for various reasons they would be more open to objection.

Book Study and Manual Training.

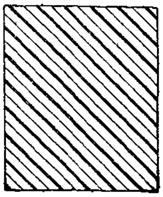
Although book study cannot pretend to compete with manual training in thoroughness, it is far more comprehensive, and, if properly handled and sufficiently extended, will teach anyone as much as he has the capacity to retain. Each illustration should be looked upon, not as a mere picture, but as representing the real thing, and the student should endeavour to realise a mental image of it as if an actual tangible solid were before him. It is this faculty of mental realisation that makes the successful architect: he not only has a knowledge of the technical detail, which anyone can master with industry, but, with only his elevations and sections before him, he has the power to form a mental image of the actual building as it will appear after completion.

The Reading of Drawings.

The expression "reading a drawing" is applied to the first elements of this faculty, and therefore every student should start by looking upon a drawing, not as a mere collection of lines, but with the perception that every line represents a visible angle or some feature of the object. In "Practical Draughtsmen's Work" (price 2s., Cassell & Co., La Belle Sauvage, London, E.C.) will be found a clear description of the principles of drawing, and a preliminary study of this little book will be of assistance in preparation for the study of building construction. It is, however, desirable go further than the mental realisation of the details illustrated in building construction.

The Use of Drawings.

The illustrations to be printed in this volume form quite a special feature from their number, their size, and their completeness of detail. A uniform system has been adopted for distinguishing the chief materials illustrated, and the accompanying blocks form the key. The illustrations throughout the book should all be redrawn, freehand or mechanically as the case may be, with the object of impressing them on the memory, and so being able to utilise them when occasion may arise at an examination or in actual building operations. Then, again, opportunity should be taken to find out the different details upon completed buildings or upon buildings in course of construction, noting the variations from standard types, and the reason for the variation, whether to reduce the cost or to increase the stability, or otherwise.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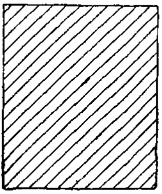
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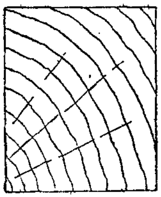
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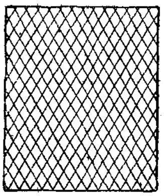
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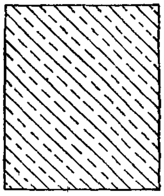
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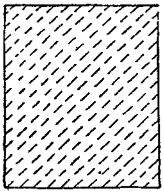
STEEL & STONE.

Figs. 1 to 6.—Pencil or Ink Section Lines for Various Materials.

General Usefulness of the Subject.

To a workman engaged in any branch of building construction, although the information in his own branch may not contain more than he is already cognisant of, the other branches will be interesting and instructive, because every workman has the possibility before him of becoming a general foreman or a clerk of works, and in such positions it is essential that he should be familiar with the principles and practice appertaining to the other trades. Then again, everyone lives in a house of some sort, and many tenants look forward to the time when they will be able to buy or build their own house, and, having obtained a general knowledge of construction, will begin to draw out plans which they hope some day to realise in bricks and mortar. A thing that is worth doing at all is worth doing well. The following chapters should therefore be read with patience and perseverance. What may be mere pastime at present may become necessary as a means of earning a living at some future date, and time spent in acquiring useful knowledge can never be wholly wasted.

Materials for Illustration.

A collection of models and samples of materials is often suggested as being useful for building construction classes, but the result of lengthened experience is that small models of construction, unless made by students for competition, are of very little practical use. They show no more than a diagram can show, and are generally incorrect in some particulars, especially in the fastenings. Special constructive arrangements, particularly about roofs and staircases, should be sought for in the buildings themselves, and all ordinary details can be found in diagrams contained in this work. Specimens of stone and timber in 4 in. cubes are very useful, with the faces prepared in different ways. Samples of bricks both whole and broken should be obtained, also samples of lime and cement. Many working tests of materials may be made without apparatus, or with only such apparatus as a student could himself very easily construct.

TIMBERS USED IN BUILDING CONSTRUCTION.

The Growth of a Timber Tree.

TIMBER trees are known botanically as *exogens* or outward growers, because the new wood is added underneath the bark outside that already formed. The whole section (Fig. 7) consists of (*a*) pith in the centre, which dries up and disappears as the tree matures; (*b*) woody fibre or long tapering bundles of vascular tissue forming the duramen or heartwood, arranged in rings, of which each one is considered to represent a year's growth, and interspersed with (*c*) medullary rays or transverse septa consisting of flat hard plates of cellular tissue known to carpenters as "silver-grain," or "felt," or "flower," and showing most strongly in oak and beech: the heartwood is comparatively dry and hard, from the compression produced by the newer layers; (*d*) alburnum or sapwood, which is the immature woody fibre recently deposited. In coniferous trees the sapwood is only distinguishable by a slight greenish tinge when dry, but when wet it holds the moisture much longer than the heartwood, and can often be detected in that way; (*e*) the bark, which is a protecting coat on the outside of the tender sapwood; it receives additions on the inside during the autumn, which cause it to crack and become very irregular in old trees. The mode of growth is as follows: In the spring the moisture of the earth is absorbed by the roots, and rises through the stem as sap to form the leaves. The leaves give off moisture and absorb carbon (in the form of carbonic acid gas), which thickens the sap. In the autumn the sap descends inside the bark and adds a new layer of wood to the tree. The actual growth is less regular than appears in Fig. 7: see Fig. 8.

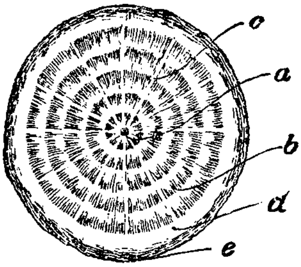

Fig. 7.—Diagram representing Cross Section of Stem of Timber Tree

Formation of Wood.

The manner in which the stem of a timber tree grows by the deposit of successive layers of wood on the outside under the bark, while at the same time the bark becomes thicker by the deposit of layers on its under side, is further illustrated in Fig. 8, which shows a cross section of an oak log. Upon examining the cross section of such trees (see Fig. 8) we find that the wood is made up of several concentric layers or rings, each ring consisting in general of two parts, the outer part being generally darker in colour, denser, and more solid than the inner part, the difference between the parts varying in different kinds of trees. These layers are called annual rings, because one of them is, as a rule, deposited every year in a manner which will be presently explained. In the centre of the wood is a column of pith *p*, from which planes, seen in section as thin lines *m*, *m* (in many woods not

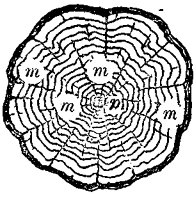

Fig. 8.—Cross Section of Oak Log.

discernible) radiate towards the bark, and in some cases similar lines *m*, *m* converge from the bark towards the centre, but do not reach the pith. These radiating lines are known as medullary rays or transverse septa. When they are of large size and strongly marked, as in some kinds of oak, they present, if cut obliquely, the beautiful figured appearance called silver-grain or felt. As mentioned under "The Growth of a Timber Tree" (p. 3), the wood is composed of bundles of cellular tubes, which serve to convey the required nourishment from the earth to the leaves.

The Function of Sap.

The action of the sap may now be described in fuller detail. In the spring the roots absorb moisture from the soil, which, converted into sap, ascends through the cellular tubes to form the leaves. At the upper surface of the leaves the sap gives off moisture, absorbs carbon from the air, and becomes denser; after the leaves are full-grown, vegetation is suspended until the autumn, when the sap in its altered state descends, by the under side of the leaves, chiefly between the wood and the bark, where it deposits a layer of new wood (the annual ring for that year), a portion at the same time being absorbed by the bark. During this time the leaves drop off, the flow of sap then almost stops, and vegetation is at a standstill for the winter. With the next spring the operation recommences, so that after a year a distinct layer of wood is added to the tree. The above description refers to temperate climates, in which the circulation of sap stops during the winter; in tropical climates it stops during the dry season. Thus, as a rule, the age of the tree can be ascertained from the number of annual rings; but this is not always the case. Sometimes a recurrence of exceptionally warm or moist weather will produce a second ring in the same year.

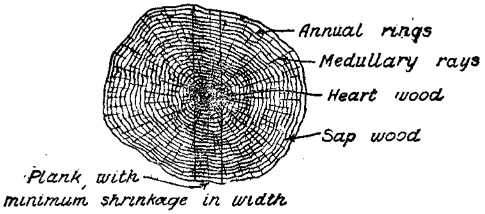

Fig. 9.—Cross Section of Oak Log showing Middle Plank.

Heartwood and Sapwood.

As the tree increases in age, the inner layers are filled up and hardened, becoming duramen or heartwood, the remainder being alburnum or sapwood. The sapwood is softer and lighter in colour than the heartwood, and can generally be easily distinguished from it. In addition to the strengthening of the wood caused by the drying up of the sap, and consequent hardening of the rings, there is another means by which it is strengthened—that is, by the compressive action of the bark. Each layer, as it solidifies, expands, exerting a force on the bark, which eventually yields, but in the meantime offers a slight resistance, compressing the tree throughout its bulk. The sapwood is generally distinctly bounded by one of the annual rings, and can thus be sometimes distinguished from stains of a similar colour which are caused by dirty water soaking into the timber while it is lying in the ponds. These stains do not generally stop abruptly upon a ring, but penetrate to different depths, colouring portions of the various rings. The heartwood is stronger and more lasting than the sapwood, and should alone be used in good work. The annual rings are generally thicker on the side of the tree that has had most sun and air, and the heart is, therefore, seldom in the centre.

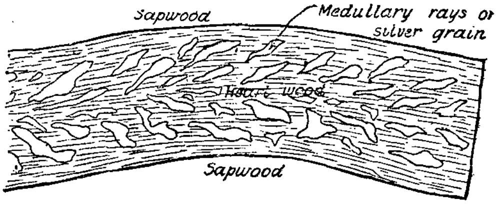

Fig. 10.—Longitudinal Section through Centre.

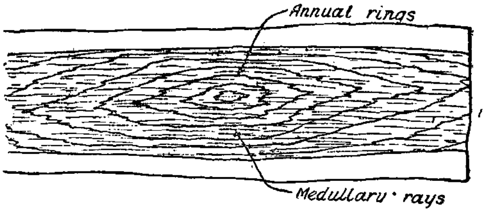

Fig. 11.—Longitudinal Section near Bark.

The Trunk of an Oak Examined.

Fig. 9 shows the cross section with the annual rings and the medullary rays, the sapwood being on the outside and the remainder heartwood. Fig. 10 shows the longitudinal section through the centre of the tree where the flower or silver-grain (that is, the medullary rays in elevation) is marked, together with the edges of the annual rings. Fig. 11 shows a longitudinal section nearer to the bark, where the graining is formed by the section of the annual rings, owing to the straight cut through the bent tree, and the broken straight lines are the sections of the medullary rays. A plank cut from the centre of the tree as shown by the straight lines in Fig. 9 would be least affected in breadth by shrinking.

Distinguishing Chestnut from Oak.

The chestnut timber used for building is the sweet or Spanish chestnut (*Castanea edibilis*), not the common horse chestnut, which is a spongy whitish wood of little use. The Spanish chestnut is only grown to a small extent in Great Britain at the present time; it may be known by the leaves being smoother, more parallel, and not radiating so decidedly from one stalk. The wood is much like oak in colour and general appearance, but when old has rather more of a cinnamon cast of colour, has less sapwood, and generally a closer grain, although softer and not so heavy as oak. The chief distinguishing characteristic of the chestnut is the absence of the distinct medullary rays which produce the flower in oak; and old roof-timbers, benches, and church-fittings may be discriminated in this way, also by the chestnut being more liable to split in nailing, while the nails never blacken the timber as they do in oak (see p. 7). Reports vary, but it seems to be decided that the roof of Westminster Hall is of oak, and that of the circular part of the Temple Church of chestnut.

Soft Woods and Hard Woods.

Soft woods have mostly distinct annual rings of growth, soft and pale in the inside, and harder and darker towards the outside, consisting of vascular tissue or woody fibre in the form of long tapering tubes, interlaced, and breaking joint with each other, having a small portion of cellular tissue at intervals, and resinous matter in the interstices. Hard woods have the annual rings much closer together, and of more uniformity in colour and hardness, but they have more or less distinct radial lines, consisting of thin, hard vertical plates, formed entirely of cellular tissue, the medullary rays or silver-grain or flower already mentioned. As regards actual hardness, the classification of mahogany, willow, yew, pitchpine, cedar of Lebanon, alder, and walnut would be :—Hard woods : Mahogany, yew, pitchpine, and walnut. Soft woods: Willow, cedar of Lebanon, alder. A classification, however, is often adopted irrespective of actual hardness, calling coniferous trees soft, and all others hard. Thus :—Hard woods : Mahogany, willow, alder, walnut. Soft woods : Yew, pitchpine, cedar of Lebanon. The weight of wood per cubic foot depends upon the specific gravity or density of the wood, and this density varies considerably in different specimens of the same kind of timber. The weights in pounds per foot cube, near enough for practical purposes, are given below in round numbers :—Spruce fir, red and white and yellow pine, larch, lime, Dantzic, Memel, and Riga, 35 lb.; chestnut, English elm, Honduras mahogany, 40 lb.; beech, birch, ash, pitchpine, American elm, 45 lb.; oak, teak, Spanish mahogany, 50 lb.; lignum vitæ and box, 80 lb.

Distinctive Characteristics of Various Woods.

American yellow deal (*Pinus strobus*), more often called American yellow pine or Weymouth pine (see p. 9), is used chiefly for panels on account of its great width; for moulding on account of its uniform grain and freedom from knots; and for patterns for casting from on account of its softness and easy working. It is very uniform in texture, of a very pale honey-yellow or straw colour, turning brown with age, usually free from knots, and specially recognised by short, dark, hair-like markings in the grain when planed, and its light weight. It is subject to cup-shakes and to incipient decay, going brown and "mothery." American woods are not branded, as a rule, though some houses use brands in imitation of the Baltic marks, described later, though without following any definite rules. The qualities may, however, very often be known by red marks I., II., III., upon the sides or ends, but the qualities of

American yellow deals are easily told by inspection, the custom in the London Docks being to stack them on their sides, so as to expose their faces to view, and allow of free ventilation. Woods from Canadian ports have black letters and white letters on the ends, and red marks on the edges.

Baltic white deal or spruce fir (*Abies excelsa*) is used in the common qualities for the roughest work—scaffold poles, scaffold boards, centering, packing cases, etc.—and in the better qualities for dressers and table tops, bedroom floor boards, cupboard shelves, etc. The wood is of yellowish white, or sometimes of a brownish red colour, becoming of a bluish tint when exposed to the weather. The annual rings are generally clearly defined, the surface when planed has a silky lustre, and the timber contains a large number of very hard glassy knots. The sapwood is not distinguishable from the heart. Baltic white deal is recognised chiefly by its small hard and dark knots, by its woolliness on leaving the saw, and by its weathering to a greyish tint. When fresh cut, the grain may be more or less pronounced than that of yellow deal. It is subject to streaks of resin in long cavities, and to loose dead knots. In white deal or spruce fir the knots are small, darker, more brittle, and opaque.

Baltic yellow deal is from the *Pinus sylvestris* or Northern pine. The colour of the wood is generally of a reddish yellow or of a honey yellow of various degrees of brightness, annual rings about $\frac{1}{10}$ in. wide, the outer part being of a bright and reddish colour. When knots occur they are from 1 in. upwards in diameter, and not very hard. This timber is stronger and more durable than white deal, the knots are of a rich red brown colour, and thin shavings of them are semi-transparent.

Elm (*Ulmus campestris*), common English elm. Characteristics: Reddish brown colour with light sapwood. Grain very irregular and numerous small knots. Warps and twists freely. Very durable if kept constantly under water or constantly dry, but it will not bear alternations of wet and dry. One peculiarity characteristic of elm is that the sapwood withstands decay as well as the heartwood. Uses: Coffins, piles under foundations, pulley blocks stable fittings, etc. Sources: Chiefly home grown; except American elm, which is a different class of wood, having straight grain and no knots.

Norwegian Timber.—Sources: Christiania, Friedrichstadt, Drontheim, Dram. Size: Average 8 in. to 9 in. square, generally tapered; scarcely called balk timber; is known as "undersized." Appearance: Much sap. Marks: on balks, others by letters, stencilled in blue on ends. Uses: Staging, scaffolding, and coarse carpentry, the best converted into deals, flooring, and imported joinery. Norwegian timber is clean and carefully converted, but is imported chiefly in the shape of prepared flooring and matchboarding. Scarce in form of yellow deals, but of high quality. Christiania best, but often contains sap. Christiania white deal used for best joinery. Christiania and Dram used for upper floors on account of white colour. Friedrichstadt has small black knots. Some Drammen deals warp and split in drying.

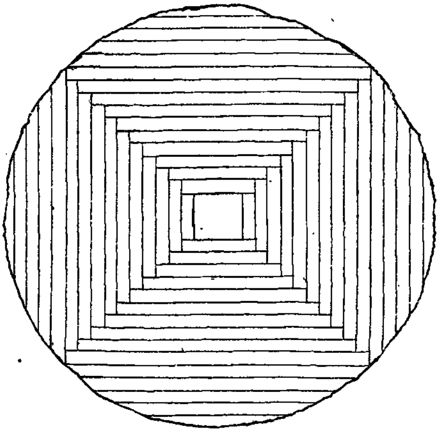

Fig. 12.—Conversion of Pitchpine Log to Show the Grain.

Pitchpine (*Pinus Australis* or *Pinus resinosa*) is recognised by its weight and strong reddish yellow grain, with distinct and regular annual rings. It must be well seasoned and free from sap and shakes. Pitchpine is very free from knots, but when they occur they are large and transparent, and give variety to the grain. It is used chiefly for treads of stairs and flooring, on account of its hardness and wear-resisting qualities; for doors, staircases, strings, handrails, and balusters on account of its strongly marked and handsome grain; for open timber roofs on account of its strength and appearance; and for outdoor carpentry, such as jetties, on account of its length and size. The ornamental grain of pitchpine is due to the annual rings, not the medullary rays as in oak, and the method of sawing oak will therefore not suit at all. The object should be to cut as many boards as possible tangent to the annual rings. About one log in a hundred will show more or less waviness of the grain owing to an irregular growth in the tree, and about one in a thousand will be worth very careful conversion. To avoid turning the log so often in cutting the boards, as in Fig. 12, they might be all cut parallel, but the result would not be nearly so good. The grain would only show straight lines towards the edges instead of a fair pattern throughout. Very careful and complete seasoning would be required, on account of the great shrinkage occurring.

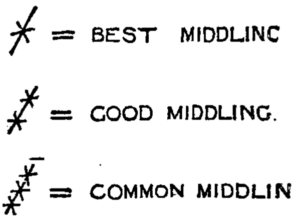

Fig. 13.—Quality Marks on Dantzic Timber.

Prussian Timber.—Sources : Memel, Dantzic, Stettin, Königsberg. The use of the balks is almost entirely confined to heavy timber work, as they are too coarse and open in the grain for carpenters' and joiners' work. They are used for outdoor carpentry and heavy woodwork, such as piles, girders, roofs, and joists. Dantzic—Size: 14 in. to 16 in. square, 20 ft. to 50 ft. long. Appearance : Subject to cup- and star-shakes and wind-cracks. Knots large and numerous, often dead and loose ; they are very objectionable when grouped near the centre of a beam, or for piles when diagonal. Annual rings wide, large proportion of sapwood (frequently the whole of the four corners of the circumscribing square), 20 ft. to 45 ft. long, heart sometimes loose and "cuppy." Marks : Scribed near centre, as in Fig. 13. Uses : Heavy outdoor carpentry, where large scantlings are required. Memel fir is tolerably free from knots, but when they occur the grain near them is irregular, and is apt to tear up with the plane.

Oak (*Quercus*).—English oak is light brown or brownish yellow, close-grained, tough, more irregular in its growth than other varieties, and heavier. Tenacity, say 6½ tons per square inch ; weight, 55 lb. per cubic foot. Baltic oak from Dantzic or Riga is rather darker in colour, close-grained, and compact ; weight, 49 lb. per cubic foot. Riga oak has more flower than Dantzic. American or Quebec oak is a reddish brown, with a coarser grain, not so strong or durable as English oak, but straighter in the grain. Tenacity, 4 tons per square inch ; weight, 53 lb. per cubic foot. African oak is not an oak at all. Exposed to the weather, oak changes from a light brown or reddish grey to an ashen grey, and becomes striated from the softer parts decaying before the harder. In presence of iron it is blackened by moisture owing to the formation of tannate of iron, or ordinary black ink.

Russian Timber.—Sources : Petersburg, Archangel, Onega, Riga, Wyborg, Narva. These yellow deals are the best for general building work, more free than other sorts from knots, shakes, sap, etc., clean hard grain and good wearing surface, but do not stand damp well. First three used for best floors—all of them for warehouse floors and staircases. Wyborg—very good, but inclined to sap. Riga—best balk timber. Size : Up to 12 in. square, and 40 ft. long. Appearance : Knots few and small, very little sap, annual rings close, wood close and straight-grained, more colour than Dantzic. Marks : Scribed at centre, as in Fig. 14. Uses : For masts and best carpentry when large enough, also for flooring and internal joinery. Petersburg—inclined to be shaky. Archangel and Onega—knots often surrounded by dead bark, and drop out when timber is worked. The Russian white deals shrink and swell with the weather, even after painting. Best from Onega. Russian deals generally come unmarked into the market, or only dry stamped or marked at their ends with the blow of a branding hammer, such marks being also termed hard brands. In some cases where the goods are not branded, the second quality have a red mark across the ends, third being easily distinguished from first quality goods. The well-known Gromoff Petersburg deals are however, marked with C. and Co., the initials of the shippers (Clarke & Company). Another good Petersburg brand is P. B. (Peter Belaieff) for best, and P. B. 2 for second quality. Timber from Russian and Finland ports, dry-stamped on the ends without colour.

Scotch fir (*Pinus sylvestris*), called also the Northern pine and red or yellow pine. From this the timber known as yellow deal is obtained; it is tough and strong for its weight, durable and easily worked, cheap and plentiful. Comes principally from the north of Europe, and is shipped at Baltic ports. Characteristics: Colour varies according to soil and habitat; generally of a honey yellow, with distinct annual rings darker and harder on the outside of each, some specimens changing to a reddish cast in seasoning, and others brownish. There are no medullary rays visible. The best has close grain and a medium amount of resin in it. Silky when planed, and when well seasoned crisp and dry to the touch. Tenacity, 5 tons per square inch; weight, 36 lb. per cubic foot. Requires periodical painting when exposed to the weather. Use: All kinds of carpentry and joinery. Source of supply: Chiefly from the Baltic ports as deals and logs.

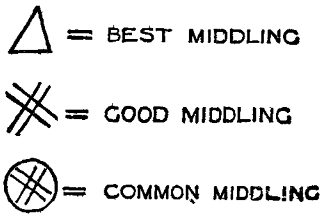

Fig. 14.—Quality Marks on Riga Timber.

Spruce fir (*Abies excelsa*), also called white deal and spruce deal or spruce. Characteristics: Has the same general character as Scotch fir, but sometimes more contrast in the grain and sometimes less. Some is very strongly marked, and other samples have scarcely any colour or distinction of grain apparent; the latter have frequently large resinous streaks, or long cavities filled with resin. The special feature is that it has numerous small dark, hard knots often black and so hard as to notch the plane-iron. Is apt to warp. Uses: The best kinds for table tops, dresser tops, upper floors, etc.; the common kinds for rough carpentry, common joinery, and packing cases. Sources: The northern ports of the Baltic, Christiania, Gothenburg, etc., whence it is imported in deals.

Swedish Timber.—Sources: Stockholm, Gefle, Soderham, Gothenburg, Sundsvall, Holmsund, Hernosand. The greater portion of this is coarse and bad, but some of the very best Baltic deal comes from Gefle and Soderham. First qualities have a high character for freedom from sap, heart-shakes, etc. The lower qualities have the usual defects, being sappy and containing large coarse knots. In the best qualities the knots are small, and larger in the lower qualities. Yellow deal, generally small, coarse and bad, with large loose knots; sappy, liable to warp and twist, but variable, best being equal to Norwegian owing to care and conversion and sorting out into different qualities. Cheap imported joinery made from these deals. Suitable for floors where warping can be prevented. Gefle and Soderham sometimes very good. White deals from Gothenburg for packing-cases, Hernosand and Sundsvall similar. Gefle and Soderham good for upper flooring, dressers, shelves, etc., and backing to veneers. There are also said to be red deals from the Baltic ports and from Canada, from the *Pinus rubra*, used for mouldings and best joinery, very like Memel. Swedish woods are never hammer-marked, but invariably branded with letters or devices stencilled on the ends in red paint, which makes it difficult to judge of their quality by inspection, as they are stacked in the timber yards with their ends only showing. Some of the common fourth- and fifth-quality Swedish goods are left unmarked, but they may generally be distinguished from Russian shipments by the bluer colour of the sapwood. The first and second qualities in Swedish deals are classed together as "mixed," being scarcely ever sorted separately, after which we get third- down to fifth-quality goods. Deals of lower quality than third are nearly always shaky, or very full of defects of some kind.

Teak (*Tectona grandis*).—The logs vary generally from 10 in. to 24 in. square, and 15 ft. to 40 ft. long. When first removed from the ship they are of a good cinnamon brown colour, but soon bleach in the sun, and might at first sight be mistaken for oak. They are stacked in piles according to the ownership, with the butt ends flush and the other ends irregular. A few business cards of the timber-broker's firm are generally nailed here and there. The balks are squared up fairly straight and true, but sometimes waney at top end with heart out of centre owing to the tree having been bent during growth. The ends are stamped with the mark of the firm, often in two or three places, initial letters in a heart, as in Fig. 15, standing for Messrs. —— Also with the number of the log, and alongside it a mark, thus ✱, or the word No. preceding the figures to show in which direction they should be read. The dimensions of the log are stamped in 1 in. figures, thus $19^3 \times 22\frac{1}{2} \times 21$ meaning 19 ft. 3 in. long by 22½ in. wide by 21 in. thick. The cubic contents are marked in red chalk, as in the strokes of Fig. 16. These are similar in composition to the quantity marks on Baltic timber, and their value is added in Fig. 16. After the logs are all stacked, the invoice mark, as $\frac{24}{2783}$, and number of the log are painted on the end of each with white paint to identify them more rapidly; and on a log near each outside the name of ship and number of pile are also painted on the end. The number of pile and name of ship are also painted at the side of each pile, on one of the logs, as in Fig. 17. The principal teak yard in London is at the South-West India Dock.

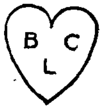

Fig. 15.—Owner's Mark on Teak.

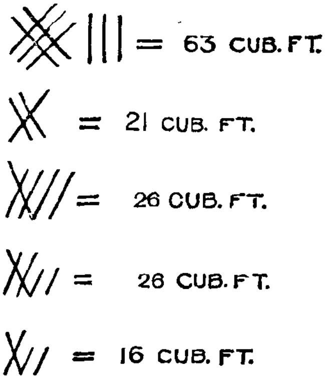

Fig. 16.—Cubic Contents of Logs.

Wainscot (*Quercus*, species doubtful) or "Dutch wainscot," a variety of oak. Characteristics: Straight grain free from knots, easily worked, and not liable to warp. In conversion it is cut to show the flower or sectional plates of medullary rays. Uses: Partitions, dados, and wall panelling generally; also for doors and windows in high class joinery. Sources: Holland and Riga, being imported in semicircular logs.

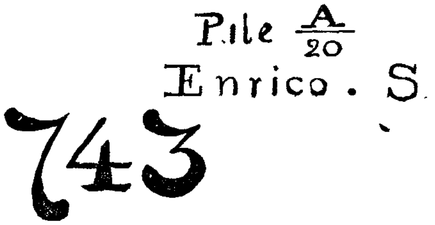

Fig. 17.—Shipping Marks.

Yellow pine (*Pinus strobus*), also called Weymouth pine, American yellow pine, American white pine, pattern - maker's pine, etc. Characteristics: Very large size, grain faintly marked, soft throughout, pale brownish yellow, almost white, changing to brown with age. Recognised always by short dark hair-like marks along the grain. Very free from knots. American yellow pine has sometimes a few small brown knots, but is generally free except at the top end, where the grain is sometimes coarse and irregular, and contains large knots. Liable to dry rot in the English climate. Takes glue well, but splits in nailing. Uses: Engineers' patterns for the foundry, wide panels for interior work, mouldings, etc. Sources: North American ports.

Selecting Timber.

Care should be taken that the timber is free from sap, large or loose knots, flaws, shakes, stains or blemishes of any kind. A light portion near one edge would indicate sap, and an absence of grain will be observed in it. This portion decays first and gets soft. The darker the natural wood, the lighter the sappy portion is usually when dry. Good timber should be uniform in substance, straight in fibre, and not twisted, warped or waney. Diagonal knots are particularly objectionable in timber for piles. If fresh cut, it should smell sweet. Has a firm bright surface, and does not clog the saw. The annular rings should be fairly regular and approximately circular, the closer and narrower the rings the stronger the timber. The colour should be uniform throughout, and not become suddenly lighter towards the edges. Good timber is sonorous when struck; a dull sound indicates decay. In specimens of the same class of timber the heavier is generally the stronger.

Specifying Timber.

All timber is to be thoroughly sound and well seasoned, free from sap, shakes, large loose or dead knots, waney edges, and other defects. That for carpentry is to be of the quality known as best middling Memel, and that for the joiners' work is to be first quality Archangel. No timber is to be fixed until it has been approved, and all rejected material is to be removed from the ground forthwith. In general for carpentry and scantling work, such as roof timbers, joists, and partitions, Memel or Riga fir (yellow deal) and Dantzic, if free from large knots. For joiners' work in general —Archangel or Onega yellow deal; for upper floors and dresser tops, Christiania white; for lower floors, Stockholm or Gefle deals; for window and door sills, oak; for stair treads where uncovered, pitchpine; for handrails, w.c. seats, bath tops, etc., mahogany; for mouldings, panels, and ornamental finishings, American yellow pine or red deal; for matchboarding, Swedish yellow battens.

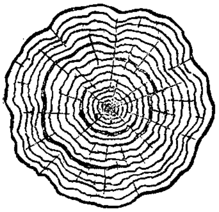
Fig. 18.—Cup-shake in Log.

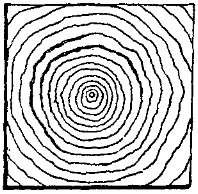
Fig. 19.—Cup-shake in Balk.

Defects in Balk Timber.

Cup-shakes. — Cracks extending circumferentially at one or more places, caused by the separation of the annual rings, as in Figs. 18 and 19.

Doatiness. — A speckled stain found in beech, American oak, and other timber, due to incipient decay. It is produced by imperfect seasoning or by exposure for a long period to a stagnant atmosphere.

Dry Rot.—If the balks have been stacked on land with insufficient ventilation, the growth of a fungus over them, like white or brown roots, may indicate that dry rot has already commenced, although it is chiefly found under kitchen floors

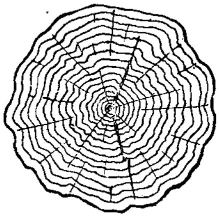
Fig. 20.—Heart-shake in Log.

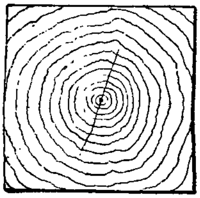
Fig. 21.—Heart-shake in Balk.

Foxiness.—A reddish or yellowish brown tint in the grain, caused by incipient decay.

Heart-shakes.—These are splits or clefts occurring in the centre of the tree, as in Figs. 20 and 21. They are common in nearly every variety of timber, and are very serious when they twist in the length, as they interfere with the conversion of the tree into boards or scantlings. They sometimes divide the log in two for a few feet from the end.

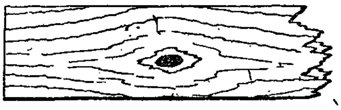
Fig. 22.—Knot.

Knots.—If large, or dead and loose, knots are objectionable, as they weaken the timber and are unsightly. In piles, if knots occur diagonally, the balk is liable to be sheared through the knots or severely damaged by the blows of the ram. (See Fig. 22.) Dantzic timber has the largest knots, spruce the hardest.

Rind-galls. — Curved swellings caused by the growth of new layers over a part damaged by insects, or by tearing off or imperfect lopping of a branch. Shown by the grain being irregular and vacuous.

Sapwood.—This occurs more in some trees than in others--say, Dantzic much, pitchpine little. May be known by its greenish tinge, and holding the water longer than the sound parts after having been wet. If creosoted, the sapwood is as lasting but not so strong as the heartwood. Generally occurs at the corners only of the balks, which arises from the desire to save as much timber as possible. (See Fig. 23.)

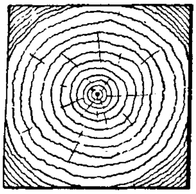
Fig. 23.—Sapwood.

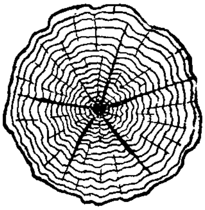
Fig. 24.—Star-shake in Log.

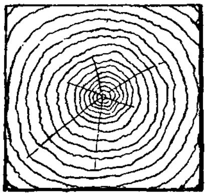
Fig. 25.—Star-shake in Balk.

Star-shakes. — When several heart-shakes occur in one tree they are called star-shakes from the appearance produced by their radiation from the centre. (See Figs. 24 and 25.)

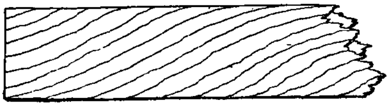
Fig. 26.—Twisted Fibres.

Thunder-shakes are irregular fractures across the grain, occurring chiefly in Honduras mahogany.

Twisted Fibres.—Caused by the tree being twisted in its growth, from the action of the wind upon the head. Not suitable for cutting up into joists or planks, owing to the fibres running diagonally in any longitudinal cut, as in Fig. 26. Oak with twisted fibres will not retain its shape when squared, but is very suitable for splitting up into plugs for bricklayers' and masons' use.

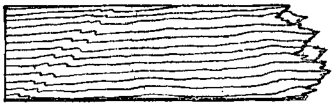
Fig. 27.—Upset.

Upsets.—Portions of the timber where the fibres have been injured by crushing, as in Fig. 27.

Waney Edges.—These occur when the top end of the tree is not large enough to hold up to the full size to which the lower end is squared. (See Figs. 28 and 29.) These balks *may be used for piling without detriment if the top end be driven downwards.*

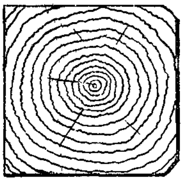
Fig. 28.—End View of Waney Edge.

Wide Annual Rings.—*These generally indicate soft and weak timber.*

Wind-cracks.—Shakes or splits on the sides of a balk of timber, caused by shrinkage of the exterior surface, as in Fig. 30.

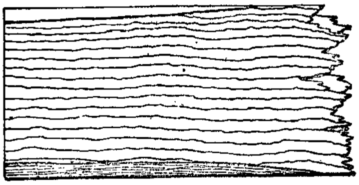
Fig. 29.—Side View of Waney Edge.

Wet Rot. — Timber that has been lying long in the timber ponds, and subjected to alternations of wet and dry, may be so soft and sodden as to have reached the stage of wet rot. The term wet rot implies chemical decomposition of the wood; whereas, as stated on p. 10, dry rot is the result of a fungous growth.

Seasoned Timber.

Seasoned timber differs from unseasoned principally in having the sap and moisture removed; this makes it drier, lighter, and more resilient or springy. It is less liable to twist, warp, or split. Artificial systems of seasoning are described in another paragraph.

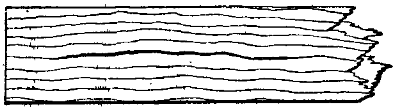

Fig. 30.—Wind-crack.

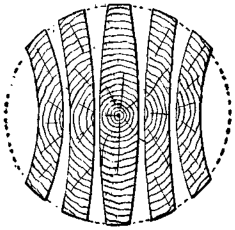

Fig. 31.—Warping of Planks according to Position in Tree.

The advantages of using seasoned timber are that it works more easily under the saw and plane, and retains its size and shape after it leaves the hands of the carpenter or joiner. When unseasoned stuff is used it warps and shrinks, and, besides being unsightly, is liable to cause failures in structures of which it may form a part; it is also very liable to decay from putrefaction of its sap.

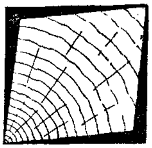

Fig. 32.—Shrinkage in Seasoning.

Methods of Converting Timber.

In converting timber into planks or boards the shrinkage and warping to be expected in use depend upon what part of the tree the piece is cut from. Practically, the stuff will only shrink along the lines of the annual rings, and not from the outside towards the centre; so that, a tree being cut into five planks, the alteration produced by seasoning is shown in Fig. 31. A piece of quartering would, in the same way if originally die-square, become obtuse angled on two opposite edges, and acute angled on the other two, as in Fig. 32. In the conversion of fir, the old system is shown by Fig. 33, which is objectionable on account of the centre deal containing the pith enclosed, and being therefore more subject to dry rot. Fig. 34 shows the modern method of conversion; where the 3×9 deals go to the English market, and the 1¼×9 to the French market. Of the remainder in each case, some is cut up into battens and fillets for slating and tiling, and similar purposes, and the rest used as fuel. In converting oak, the method will depend upon the purpose for which it is required. For thin stuff, where the silver grain or "flower" is desired to appear, the

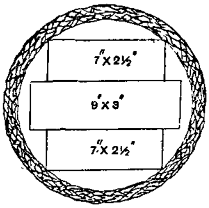

Fig. 33.—Old Method of Converting Logs into Deals.

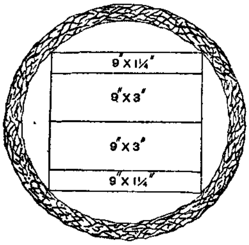

Fig. 34.—Modern Method of Converting Logs into Deals.

method shown at A (Fig. 35) is best, and that at B second best, the object being to get the greatest number of pieces with the face nearly parallel to the medullary rays. The method shown at C makes less waste, but does not show up the grain so well; while D is the most economical when larger scantlings are required.

Methods and Effects of Seasoning.

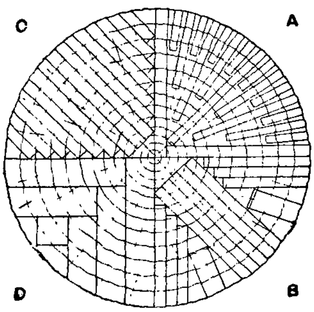

Fig. 35.—**Conversion of Oak into Boards.**

Timber cut down in the autumn, after the sap has formed the new layers of wood, is best seasoned by cutting it into planks and stacking them *horizontally* in open order under cover. *exposed to a free current of air, and protected* from ground moisture. Hard woods are generally stacked with thin strips between them, placed transversely every 4 ft. or so, and soft woods by laying them on edge with spaces between, the direction being crossed in adjacent courses. Time occupied, say, two years. Balk timber is best seasoned by putting it under water in a running stream for a few weeks, then stacking it loosely with some protection from sun and rain. These are termed natural processes. There are various artificial processes of seasoning in use which expedite the work and shorten the time necessary between felling and using, but the strength and toughness of the timber are reduced. The methods are—desiccating, or using hot-air chambers, smoking, steaming, and boiling. To reduce the risk of splitting the ends in the drying process, they are clamped—that is, thin pieces are nailed over the end grain so that the ends may dry uniformly with the other parts. McNeile's process is said to be very good: the wood to be seasoned is exposed to a moderate heat in a moist atmosphere charged with the products of combustion, say CO_2, which is supposed to convert the sap to woody fibre and drive out the moisture. Smoke drying over an open wood fire drives out the sap and moisture and renders the wood more durable and less liable to attack by worms. During seasoning a large proportion of the moisture evaporates, causing the fibres to shrink and the timber to become less in bulk and weight. It also becomes lighter in colour and more easily worked. The shrinkage is scarcely perceptible in the length, but is very considerable in the width, measuring circumferentially on the annual rings. Radially, or in the direction of the medullary rays, the shrinkage is only slight. If the log is whole, the shrinkage causes shakes and wind-cracks; if cut up into planks or quartering, the shrinkage is determined by the position of the annual rings, and, with care, no shakes are caused. Timber is considered fit for carpenters' work when it has lost one-fifth of its weight, and for joiners' work when it has lost one-third.

How Baltic Flooring-boards are Made.

The log is first slabbed, or the two outside pieces are cut off, at an ordinary saw-bench, from a log, say, 10 in. diameter, leaving a thickness between of, say, 7¼ in. The log is then placed upon a saw-bench with a jig containing six band-saws spaced for cutting it into the required thicknesses of, say, 1⅜ in. The boards are then passed through a planing machine that can true up either one face and one edge, or both faces and both edges at the same time, converting each into a batten 7 in. by 1¼ in. Or the faces alone may be done first, and the edges may be done separately by passing the boards between a pair of vertical revolving cutters forming a groove in one edge, and leaving a projecting tongue on the other edge, the remainder of the edge being trued up to a flat surface by the same cutters.

Dry Rot: Its Cause, Cure, and Prevention.

Dry rot is a special form of decay in timber, caused by the growth of a fungus, *Merulius lachrymans*, which spreads over the surface as a close network of threads, white, yellow, or brown, and causes the inside to perish and crumble. Various causes may combine to render the timber favourable to the growth of this fungus—namely, large proportion of sapwood; felled at wrong season when full of sap; stacked for seasoning without sufficient air spaces being left; fixed before thoroughly seasoned; painted while containing moisture; built into wall without air space; covered with linoleum; exposed to warm stagnant air, as under kitchen floors. There is no cure when the fungus has obtained a good hold. The worst must be cut out and remainder painted with blue vitriol (cupric sulphate). The best preventive is to use only well-seasoned timber and to provide for its due ventilation in the structure.

Conditions Favourable to Dry Rot.

The conditions favourable to dry rot, including some that have been already indicated, may be thus summarised:—(*a*) As regards the timber itself: If it contains a large proportion of sapwood; if too young or too old when cut down; if cut down in the spring or the fall of the year instead of in midwinter or midsummer, when the sap is at rest. (*b*) As regards its treatment: If stacked for seasoning without air spaces; if fixed before thoroughly seasoned; if painted or varnished while still containing moisture. (*c*) As regards surrounding conditions: If built into wall, covered with linoleum or kamptulicon, or otherwise excluded from free access of air; if exposed to warm, damp, and stagnant air, as under kitchen floors; if liable to contact with germs of the fungus from other pieces.

Detection and Treatment of Dry Rot.

When dry rot is suspected in a floor the floor-boards should be lifted at the corners of the room, or at dead ends of passages, or wherever signs of weakness show themselves, and the surfaces of the joists, wall-plates, and under side of the floor-boards should be closely examined for fungus, mildew, or any unhealthy sign, such as a brown semi-charred appearance. If any is found, the worst parts should be cut out and renewed, the remainder well scraped over, including the walls, and well washed with a solution of blue copperas (sulphate of copper). If the earth below is found to be damp, a layer of cement concrete should be spread over it, not less than 4 in. thick. Air bricks and ducts should be placed in the walls on opposite sides, to get a through current, as moist, warm, stagnant air is the most potent aid to dry rot; and every endeavour should be made to obtain thorough ventilation. The means of prevention are: Thorough seasoning, free ventilation, creosoting or charring if necessarily exposed to damp earth, and painting with vitriol or cupric sulphate.

Preservation of Wood Underground.

The best way to preserve wood from decay when buried in the ground is to creosote the wood; this does not mean painting the wood over with tar, but proper creosoting by the regular process. The butt-end of a post to be placed in the ground may be charred over a wood fire, quenching with water when the wood is charred say ¼ in. to ½ in. deep. This will prevent rotting and the attacks of worms, but it is necessary that the wood should be previously well seasoned, or the confined moisture will cause decay. Chloride of zinc and water, about 1 to 4, in which wood is steeped under Sir Wm. Burnett's system, preserves the timber from decay and renders it incombustible. A method sometimes adopted is to bed the posts in cement concrete, but this is not quite so good as creosoting.

Ascertaining Strength of Timber.

The machines used for testing the tensional, compressional, and other strengths of timber and other materials are very elaborate and very expensive, as, unless the experiments were efficiently carried out, they would be worse than useless. In testing for tensile strength, the piece of timber may be from ½ in. to 3 in. square, held between toothed jaws, or shouldered and held between clips, but it is essential that the stress should be direct, that is, in the true axial line of the piece. The same sizes may be used for testing compressive strength, the ends being made perfectly true and square, and not shouldered. Timber is, however, more often tested for transverse strength, and home experiments may be made which will give a rough approximation. What is wanted is to find a value for C in the formula $W = \frac{C\,b\,d^2}{L}$, where W is the breaking load, C a coefficient varying with the material and the mode of loading and supporting, *b* the breadth in inches, d^2 the depth in inches squared, and L the clear span in feet. If the piece be simply supported at both ends and loaded in the centre, C will be about $3\frac{1}{2}$ cwt. or 400 lb. for fir or deal. Say a piece of straight yellow deal, $\frac{3}{4}$ in. square and 3 ft. long, is carefully prepared and laid across two supports fixed level at a distance of 24 in. from each other, and an empty galvanised iron bucket hung on the centre of the beam. Then the bucket can be gently filled with dry sand until the small timber beam cracks and breaks. It can be arranged that the bucket does not fall far, and then the bucket and sand can be carefully weighed. Suppose it to be 80 lb., then the calculation will be $80 = C \times \frac{\cdot 75 \times \cdot 75^2}{2}$; $80 = C \times \cdot 2109$; $\therefore C = \frac{80}{\cdot 2109}$ = say 380 lb. or $\frac{380}{112}$ = say 3·4 cwt.

Weight and Strength of Timber.

These particulars are given in the accompanying table, in which the columns are marked as follows :—(A) Name of timber, selected quality; (B) weight lb. per cubic foot; (C) ultimate tensile strength tons per square inch; (D) ultimate compression tons per square inch; (E) coefficient of transverse strength; (F) ultimate bearing pressure tons per square inch across grain.

A	B	C	D	E	F
American red pine - -	37	—	2·2	4·0	—
Ash - - - - - -	45	2·0	3·5	5·0	—
Baltic oak - - - - -	48	3·0	3·2	4·3	—
Beech - - - - - -	47	1·9	3·8	4·5	—
Elm - - - - - -	37	2·0	3·0	3·0	—
English oak - - -	50	3·0	3·2	5·0	·90
Greenheart - - - -	60	—	5·8	8·0	—
Honduras mahogany - -	35	1·5	2·8	4·9	·58
Kauri pine - - - - -	38	—	2·8	4·8	—
Larch - - - - - -	35	1·5	2·5	3·5	—
Northern pine - - -	37	1·5	2·9	4·0	·60
Pitchpine - - - - -	50	—	2·9	5·0	·76
Spanish mahogany - -	53	1·8	3·0	5·0	1·9
Spruce fir - - - - -	31	1·5	2·5	3·6	·22
Teak - - - - - -	50	3·0	3·8	5·0	—
White pine - - - -	28	—	1·8	3·8	·27

The safe load in tension and compression, columns C and D, would be from one tenth to one-fifteenth of the amounts given. The safe bearing pressure across the grain of timber as at the ends of a beam will be about one-fifth of the amounts given in column F. Column E gives the coefficient C in the formula $W = C\, b\, d^2 \div L$, and the safe load would be about one-sixth of W for temporary work, or one-tenth for permanent loads.

Cutting the Strongest Beam from a Round Log.

By mathematical investigation Fig. 36 shows the graphic method of finding the strongest rectangular beam that can be cut out of a round log of timber. The diameter is divided into three equal parts, and perpendiculars are raised on opposite sides on the inner ends of the outer divisions. The four points in which the circumference is touched are then joined to give the beam A E B F. The proportion is $\frac{FB}{AF} = \frac{1}{\sqrt{2}}$, because by Euclid II. 14, $\sqrt{AD \times DB} = FD$, and by Euclid I. 47, $FB = \sqrt{(FD)^2 + DB^2}$ also $AF = \sqrt{(AB)^2 - (FB)^2}$. Let $AB = 1$, then $FD = \sqrt{\frac{2}{3} \times \frac{1}{3}} = \frac{\sqrt{2}}{3}$, $FB = \sqrt{\left(\frac{\sqrt{2}}{3}\right)^2 + (\frac{1}{3})^2} = \sqrt{\frac{2}{9} + \frac{1}{9}} = \sqrt{\frac{1}{3}} = \frac{1}{\sqrt{3}}$, $AF = \sqrt{1^2 - \frac{1}{3}} = \sqrt{\frac{2}{3}} = \frac{\sqrt{2}}{\sqrt{3}}$ and $\frac{FB}{AF} = \frac{1}{\sqrt{3}} \div \frac{\sqrt{2}}{\sqrt{3}} = \frac{1}{\sqrt{2}}$. When the diameter of log $AB = d$, the depth $AF = \frac{\sqrt{2}}{\sqrt{3}} = \frac{1 \cdot 414}{1 \cdot 732} = \cdot 816d$, and the breadth $FB = \frac{1\cdot}{\sqrt{3}} = \frac{D}{1 \cdot 732} = \cdot 577d$. This, of course, only shows the mathematical calculation corresponding to the graphic diagram, and does not in any way prove the statement that this beam will be the strongest that can be cut out of a round log, which would probably be a laborious matter. But given such a beam, its strength could be calculated by ordinary formula, and then another beam slightly narrower, and a beam slightly broader, both inscribed in the circle, could be tested by the same formula.

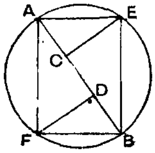

Fig. 36.—Strongest Beam from a Round Log.

Cutting the Stiffest Beam from Round Log.

The stiffest rectangular beam that can be cut out of a round log of timber is shown in Fig. 37, where the diameter is divided into four equal parts, but otherwise the construction and calculation will be on similar lines to the above, resulting in $\frac{FB}{AF}=\frac{1}{\sqrt{3}}$; and with a log of diameter $AB=d$ the depth of the stiffest beam will be ·866d and the breadth ·5d.

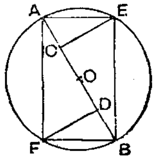

Fig. 37.—Stiffest Beam from a Round Log.

Timbers for Various Purposes.

In the following list the timbers are stated in order of superiority for the purposes named. All the timber should be specified according to the precise quality required, and not merely as "the best."

Dock Gates. — Greenheart, oak, creosoted Memel. The specification of the 60 ft. entrance lock gates at the Victoria Dock, Hull, provided for ribs, heads, and heels of single squared timbers either of English oak of the very best and quickest grown timber or of African oak, but no mixture of the two. The planking was specified to be of greenheart.

Floor Boards.—Oak, pitchpine, Stockholm or Gefle yellow deal; and for upper floors, Dram or Christiania white deal. For common floor-boarding, Swedish or Norwegian yellow or white deal.

Floor Joists.—Russian deals make the best joists, as they are straight-grained and free from knots, sound and tough. Baltic fir is cheaper and next best. Swedish and Norwegian not reliable.

Half-timber Framing.—Oak is best, as it resists decay the longest, and can be obtained naturally shaped in curves or straight, as may be required. The colour and texture are also suitable for architectural effect. Teak is good, but does not weather quite so good a colour; it is apt to split with nailing. Larch is next best.

Pile Foundations. — Greenheart, oak, elm, creosoted Memel, alder. Greenheart is undoubtedly best, but the cost is prohibitive except for marine work, where it is sometimes essential, as sea-worms will not attack it. Oak is next best when it can be afforded. Memel fir (*Pinus sylvestris*) in 13 in. to 14 in. whole timbers, creosoted or in its natural state, is the most suitable under ordinary circumstances, owing to its convenient size, length, and general character. Riga fir is generally too small, and Dantzic fir too large and coarse. Pitchpine is considered suitable by some; its chief advantage is the large size and great length in which it may be obtained. American elm and English elm, beech, and alder are suitable if wholly immersed, but not otherwise.

Planking to Earth Waggons.—Elm, with ash for shafts, if any.

Roof Trusses. — Oak, chestnut, pitchpine, Baltic fir (Dantzic, Memel, or Riga). For tie-beams to open timber roof 40 ft. span pitchpine is best, as it can be obtained free from knots, in long straight lengths, and the grain is suitable for exposure either plain or varnished. Oregon pine is suitable for similar reasons, but not so well marked in the grain. Riga fir is good material for roof timbers, but difficult to obtain in long lengths. For tie-beam of king-post roof truss, the same as above, or pitchpine (*Pinus Australis*), if it is to be wrought and varnished.

Shop-fronts.—For external doors to a bank well-seasoned English oak. For internal four-panel door, yellow deal from Christiania, Stockholm, or Gefle. When free from knots this is often called red deal. For a shop-front, Honduras mahogany (*Swietenia mahogani*) or American walnut (*Juglans regalis*).

Treads of Stairs. — Oak, pitchpine, Memel fir, ordinary yellow deal.

Weather-boarding. — Oak is best under all circumstances, but is expensive. Larch (*Larix Europœa*) perhaps stands next, as it resists the weather well and bears nails without splitting. Ordinary weather-boarding consists of yellow deal from various ports—say four out of a 2½ in. by 7 in. batten or 3 in. by 9 in. deal cut featheredged. For work to be wrought and painted American red fir is clean-grained and cheap. For very common rough work white spruce deal may be used as being the cheapest.

Window-sills.—Oak.

Cubing Round Timber.

To get the solid contents of round timber :—Take one-fourth of the middle girth of the timber in inches, square this dimension, multiply by the length in feet, and divide by 144 to obtain the reputed cubic contents. If the bark is on, make an allowance for it by deducting 1 in. per foot from the actual girth before dividing by 4. Example :—Round log of oak 20 ft. long, 18 in. diameter one end and 12 in. the other, girth 48 in. Then 48 in. = 4 ft., 1 in. per foot = 4 in., and 48—4 = 44 in. ; quarter girth = 11 in., 11 squared = 11 × 11 = 121, and 121 × 20 = 2,420. Then $\frac{2,420}{144} = 16·8$, say 17 cub. ft.

Dimensions of Sawn Timber.

Battens is the name given to narrow floor-boards 7 in. wide when unwrought, or 6¾ in. wide when wrought. Floor-boards 4½ in. wide are called narrow battens. Slate battens are strips, 3 in. by 1 in. or other size, fixed to rafters for carrying slates. Batten is also a general term for any narrow strip of wood.

Deals are boards 9 in. wide and generally 3 in. thick.

Planks are boards 11 in. wide and generally 3 in. thick.

Scantling timber is usually understood to be squared timber from about 4 in. to 12 in. side, although timber from 2 in. to 6 in. side is often called quartering ; but any piece of timber not being part of a plank, deal, or batten, when sawn all round, is called a piece of scantling, whether the timber is sawn die-square (that is, equal sided) or not. A brief definition of scantling is "Timber sawn up into the usual market sizes as distinguished from timber in log."

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IRON AND STEEL USED IN BUILDING CONSTRUCTION.

Characteristics of Iron and Steel.

THE essential difference between cast iron, wrought iron, and steel consists in the relative amount of carbon each contains. Say wrought iron, ¼ per cent.; mild steel, ¼ to ½ per cent.; cast steel, ¾ to 1½ per cent.; cast iron, 2 to 6 per cent. Cast iron is crystalline and brittle, cannot be forged or welded, and is six times stronger in compression than in tension. Wrought iron is tough, malleable, and ductile, fibrous, can be forged and welded, and is of nearly equal strength in tension and compression. Cast steel is like cast iron, but much stronger; mild steel is like wrought iron, but more homogeneous, softer and tougher. They can be distinguished one from another by the following method: Hold a piece in the fire by the smith's tongs; when it is a good full red, strike it with a hammer on the anvil, and if cast iron it will fly to pieces. If it does not, plunge it in water to cool it suddenly. If it is hardened it is steel; if it is not hardened it is wrought iron. Practical tests for iron and steel are detailed on p. 21. Malleable cast iron is made by heating ordinary castings, preferably of white cast iron, for two to forty hours, according to size, in contact with oxide of iron or powdered red hæmatite, causing partial conversion into wrought iron by abstraction of carbon.

Pig Iron.

Pig iron is classified under the heads of Bessemer iron, foundry iron, and forge iron. Bessemer iron is a variety of pig iron made from hæmatite ores for conversion into steel, and very free from impurities. Foundry iron is all pig iron having grey fracture and a large proportion of uncombined carbon; produced under high temperature and a full supply of fuel. Forge iron is white pig iron, almost free from uncombined carbon; suitable for conversion into wrought iron, and produced with low temperature or insufficient fuel, frequently run from the blast furnace into iron moulds, rendering it brittle for ease in breaking up. Foundry pig is further classified. No. 1 pig is chiefly used in the foundry; colour dark grey, crystals large and leafy, carbon in form of graphite; very soft, melts very fluid, but being coarse-grained will not give a sharp impression; cools slowly. For fine castings the presence of a little phosphorus is advantageous; the grain is finer, the iron a lighter colour, and the impression sharper; used for small castings, hollow ware, small machinery, etc. No. 2 pig is grey and mottled in colour; carbon partly combined; used for large castings in dry sand or loam; melts fluid, is tough, of close texture, fills the mould well, more free from impurities than No. 1. Heavy machine castings are made from No. 2, or various mixtures of Nos. 1, 2, and 3. No. 3 pig is hard and white, used for mixing; carbon most chemically combined.

Cast Iron.

The characteristics of cast iron, some of which have been already indicated, may now be summarised. It is crystalline, brittle, fluid at high temperatures, melting at 2,200° F. to 2,750° F., and taking complicated shapes by casting in moulds; contains 2 to 6 per cent. of carbon, part mechanically combined and part chemically. No. 1 quality is soft and dull grey, has most carbon mechanically combined; No. 3, hard and silvery white, has most carbon chemically combined; No. 2, mottled, is intermediate in character and appearance. No. 1 is used for small fancy castings, No. 2 for machine castings where strength is required; No. 3 for mixing only. Cast iron is six times stronger in compression than in tension, and is used chiefly for parts subject to dead load only, such as columns, base-plates, and for shaped articles such as brackets, gutters, etc. Iron containing a large amount of free carbon would be known as grey cast iron ; it would be very soft, adapted for general use in the foundry, colour dark grey, crystals large and leafy, with visible black specks of uncombined carbon ; melts very fluid. With the same amount of carbon in chemical combination it would be known as white cast iron ; it would be very hard and brittle, with brilliant silvery crystalline fracture, not fit to use by itself in the foundry ; melts at a lower temperature than grey cast iron, but runs less freely.

Hot-blast and Cold-blast Cast Iron.

Cast iron may be divided into hot-blast iron and cold-blast iron, so named from the temperature of the blast used in smelting the ores. Hot blast is generally quicker and more economical, requiring only 30 cwt. of coke per ton of metal instead of 40 cwt., but the metal is not considered to be so strong. It is difficult to distinguish the two varieties, but, other circumstances being equal, hot blast has rather a finer grain, duller fracture with sometimes patches of coarse grains, and usually more impurities. Increasing the blast or reducing the supply of fuel makes the iron whiter, harder, and less suitable for re-melting, but better for conversion into wrought iron or steel. Temperature of blast from 600° F. to 1,000° F., but higher temperatures have been attained in the Cleveland district.

Wrought Iron.

Wrought iron is fibrous, tough, soft, ductile at high temperatures, but not fluid, may be pressed to shapes in moulds at 1,500° F. to 2,000° F., welded at 2,500° F. to 2,800° F., and forged, hammered, or rolled to various shapes. It contains not more than 0·25 per cent. of carbon, and readily oxidises ; is of nearly equal strength in tension and compression, and is used for rolled sections, boilers, tie-rods, chimney bars, bolts, nuts, and rivets. Merchant bar is the commonest form of wrought iron that can be used as such ; it is very hard and brittle, and can only be forged with difficulty. It is made by piling up short lengths of puddled bar, raising them to a welding heat, and passing them through rollers. This unites them into a single bar and gives the iron a fibrous structure, which greatly increases its strength. Best bar is the material after having gone through this process a second time. It is tougher and more easily worked than merchant bar, and is used for general construction.

Steel.

Steel in general is intermediate between cast and wrought iron, fibrous to crystalline ; when containing a small amount of carbon may be welded, and with more carbon may be cast. Is very tough and strong, and can be forged and tempered ; and is used for boiler and bridge plates when containing little carbon, and for tools when containing more carbon. Steel may be made by the addition of carbon to wrought iron, or the abstraction of carbon from cast iron. Both methods are in use commercially, but the old classification by which the percentage of carbon alone determined the designation is now nearly discarded, and the better definition of steel would seem to include "all those malleable forms of commercial iron containing iron and carbon produced from a state of fusion into a malleable ingot." When the carbon contained is less than 0·5 per cent., the result is "mild steel," although specimens of wrought iron may be found containing a higher percentage of carbon. It is very tough and ductile, with little rigidity or elasticity. With care it can be forged and welded like wrought iron. Steel containing more carbon is less ductile but stronger and tougher, and, although brittle, can be tempered for cutting tools, but is forged with difficulty, and cannot be welded.

Manufacture of Wrought Iron.

White pig iron or forge iron, almost free from uncombined carbon, is melted with coke or charcoal in an open hearth or "refinery furnace" supplied with an air blast so as to impinge on the melted metal and furnish an oxidising atmosphere. This carries off a portion of the carbon, and at the same time removes a portion of the impurities, particularly silicon, in the form of slag. The melted metal, having lost some of its carbon, is then poured into a cast iron trough lined with loam, kept cold by water circulating below, and the sudden chilling has the effect of converting soft grey iron into hard silvery white metal, the carbon which formerly existed in the shape of graphite entering into perfect chemical composition. By this change the fluidity of the iron is reduced, and the subsequent puddling process facilitated. For common wrought iron the pig metal goes direct to the puddling furnace without undergoing the intermediate refining. There are two systems of puddling in use—dry puddling and wet puddling.

Dry Puddling of Iron.

Dry puddling is the process of obtaining wrought iron by burning the carbon out of refined cast iron in a reverberatory furnace. The oxygen of the air, at the high temperature employed, combines with the carbon to form carbonic acid gas, which escapes, and combines with the silicon to form silica, which runs off as slag. In hand puddling the mass is stirred about until it is of sufficient tenacity to be lifted out of the furnace in balls or blooms of 60 lb. to 80 lb. each; a 5-cwt. charge takes about two hours to work off. In Danks' rotary furnace the revolution of the furnace mechanically effects the same purpose as hand labour. If the operation be stopped before the carbon is all removed, puddled steel is obtained.

Wet Puddling of Iron.

Wet puddling or pig boiling is the more modern process, in which grey unrefined pig iron is converted direct. The bed of a reverberatory furnace is lined with broken slag, cinder, scale, etc., fused together, and over these a fettling of soft red hæmatite or "puddlers' mine" is placed. The stages of the puddling process are: (1) Graphitic carbon converted into combined carbon, and silicon partly oxidised by roasting and melting; (2) metal drawn from sides and mixed with that in centre; (3) metal "boiled" for twenty minutes, impurities being oxidised by agitation of the mass; (4) pasty metal "balled" and reballed, ready for shingling. After removal from the puddling furnace, at a welding heat, the blooms are put under a heavy trip hammer, a rotary squeezer, or a hydraulic press, to remove the slag and impurities from the spongy mass, and to solidify the metal. Puddled bar is the material after passing a bloom through the first series of rolls.

Making Merchant Bar Iron.

As stated under "Wrought Iron" (p. 19), merchant bar is made by cropping, piling, reheating, welding, and rolling puddled bar. Single, double, and treble best signify the number of times the material is again put through these processes. If the iron is overheated in any of the processes after puddling, it will be "burnt" and brittle, besides showing dark streaks and patches when machined or filed. Cold rolling of iron bars and plates increases their density and tenacity, and puts a planished surface upon them, which is useful in special cases, but lessens their ductility. In the refining process, if the sulphur is not all removed, the iron will be red short—that is, brittle when hot and difficult to forge. If the phosphorus is not removed, it will be cold short, or brittle at ordinary temperatures, although it will forge all right. If the blooms are not properly shingled, slag will be shut in and produce laminations in the rolled bars; and if the bars are allowed to lie about and get dirty before undergoing the subsequent processes, lamination will occur.

Making Best Bar Iron.

Staffordshire best is produced by cutting, piling, heating, welding, and rolling merchant bars, being the third quality produced in the order of manufacture. It becomes tougher and more fibrous, and can be forged better than merchant bar. It is used for all common purposes. Double and treble best signify repetitions of the process, and the material produced is more suitable for good work. To distinguish them—(1) When broken under a slowly applied tensile strain. Ultimate tensile strength: Merchant bar 18 tons per square inch, elongation 2½ per cent. Ultimate tensile strength: Staffordshire best 20 tons per square inch, elongation 6 per cent. (2) When nicked on one side and bent double, merchant bar would show the greater part crystalline; the other would have a more fibrous appearance with very little crystal. (3) When nicked all round and broken across, merchant bar would appear entirely crystalline; the other would have some fibre showing, although chiefly crystalline.

Blister Steel.

Blister steel is produced by a process called cementation. Bars of purest wrought iron are placed in a furnace between layers of charcoal powder, and kept at a high temperature (say 1,400° F.), for from five to fourteen days. The bars are now brittle, crystalline, and more or less covered with blisters. Small regular blisters and fine grain denote good quality. Used for facing hammers, etc., but not for edge tools; also used largely for conversion into other kinds of steel.

Bessemer Steel.

Bessemer steel is made from grey pig iron containing a large proportion of free carbon, and a small quantity of silicon and manganese, and free from sulphur and phosphorus. Iron is melted in a cupola and run into a converter lined with fire-brick and suspended on hollow trunnions. Air is then blown through the metal for about twenty minutes to remove all carbon; 5 to 10 per cent. spiegeleisen is then added, and the blowing resumed long enough to incorporate the two metals; the steel is then run out into moulds. Ingots, being porous, are reheated and put under the steam hammer, then rolled or worked as required. Used for rails, tyres, common cutlery and tools, roofs, bridges, etc.

Practical Tests for Iron and Steel.

Additional practical tests by which to decide whether a given bar is of steel or wrought iron may now be given. A rough test is described on p. 18. Look at the ends, and, if cut cold, wrought iron will show coarse fibres and possibly some crystals; mild steel will show a finer grain and more appearance of tenacity. If sheared, the steel will show more compression from the pressure of the fixed knife, due to the greater toughness. Support the bars in a sling of string, and strike them with a mallet or billet of wood—the steel will have a clearer ring. File or grind a small place to a bright surface, and put a drop of dilute nitric acid on it—the steel will show a darker stain. Put an equal weight of filings from each into diluted nitric acid, and the steel will colour the liquid darker. The steel will be tougher to chip with a chisel than the wrought iron, and will stand a higher tensile stress with more elongation. It will also bend cold to a greater angle without cracking. By holding the steel in the "magnetic dip," and striking a blow on the end, permanent magnetism will be induced, while wrought iron will not retain it. An iron bar that has been broken by a slowly applied tensile stress then. (*a*) Supposing it to be wrought iron, the fracture would be fibrous with points more or less projecting, and sometimes a step in the line of fracture with dirt on its side. A small part may show bright crystals. The piece would elongate before fracture, and the sectional area would be contracted at the point of fracture. (*b*) Supposing it to be cast iron, the fracture would take place suddenly, and would show a dull crystalline irregular surface of uniform colour; there would be very little elongation, and no appreciable contraction of area. If the stress had been suddenly applied, the difference in the case of the wrought-iron bar would be that the elongation and contraction of area would be less, and a larger portion would be crystalline. The effect of heating and suddenly cooling bars of cast iron, wrought iron, and steel is that the cast iron is excessively hard and apt to fly to pieces, and the wrought iron is unchanged, while the steel becomes brittle and varies in hardness according to the amount of carbon it contains.

Defects in Cast Iron.

The principal defects in cast iron are: (*a*) It may be too hard for the required purpose, which is found by trying it with a file or by striking the edge with a hammer, when a fragment will fly off and the fracture will be whitish grey. (*b*) It may be too soft, which is found by trying with a file or by striking the edge with a hammer, when an indentation will be made, and the surface of a fracture will be a dull dark grey. (*c*) It may be honeycombed, when it will show holes in the surface more or less grouped, especially on the upper side. (*d*) It may be run too cold, when the surface will show lines of indentation, or cold-shuts. (*e*) It may contain too much carbon when parts may be so soft as to be cut with a knife. (*f*) It may be badly mixed, containing pieces of burnt scrap, when the casting will machine irregularly, and hard lumps will be found. (*g*) Good scrap, but of different qualities, may have been used, causing an irregular surface of swells and hollows, owing to the unequal shrinkage in cooling. (*h*) Fractures may be seen in a new casting from initial strains in cooling, owing to improper designing. Cast-iron columns should be of a strong metal, produced by a mixture of No. 1 and No. 2

pig metal. They should be sound, and free from scale, scabs, cold-shuts, fins, honeycomb, etc., and true to dimensions. Test bars 3 ft. 6 in. long, 2 in. deep, and 1 in. wide should be separately cast, three from each melting at which the columns are run. The lower side or thin edge should be placed downwards upon rigid bearings, 3 ft. apart. With a load of 25 cwt. in centre, having a bearing of not more than 1 in. wide, the bars should deflect not less than $\frac{3}{10}$ in., and should break with a minimum load of 28 cwt., and an average for three bars of not less than 30 cwt.

Defects in Steel.

The principal defects in cast steel are : (*a*) (*c*) or (*h*) as above. The principal defects in mild steel are : (*a*) Too hard, owing to an excess of carbon. (*b*) Laminations, owing to honeycomb in the ingot being rolled out into the bar or plate. (*c*) Loss of strength, owing to being worked at a blue heat without subsequent annealing. (*d*) Loss of strength, owing to presence of sulphur or phosphorus. Riveted steel girders should be true to dimensions, with the requisite camber. The rivets should be equally spaced according to drawings, truly in line, with full-sized heads and the snap or cupping tool should not have cut into the plates. The edges of the plates should be close together, and the butting ends on the compression side should be machined and perfectly close. The material should be tested for tensile strength and bending. The plates should have an elastic limit of not less than 14 tons, and the bars and rivet rods of 15 tons per square inch. The ultimate strength should be not less than 28 and 30 tons respectively, and should not exceed 32 tons per square inch. The elongation in a length of 10 in. should not be less than 20 and 25 per cent. respectively. The plates should also be capable of bending to an inside radius of one and a half times their thickness when heated to a low cherry red and cooled in water of a temperature of 82° F. The rivet rod should bear doubling close while cold without fracture or hair cracks.

Testing Iron Castings.

The best and most certain test of the quality of a piece of cast iron is to try any of its edges with a hammer ; if the blow of the hammer makes a slight impression, denoting some degree of malleability, the iron is of good quality, provided it be uniform ; if fragments fly off and no sensible indentation be made, the iron will be hard and brittle. The utmost care should be employed to render the iron in each casting of a uniform quality, because in iron of different qualities the shrinking is different, which causes an unequal tension among the parts of the metal, impairs its strength, and renders it liable to sudden and unexpected failures. When the texture is not uniform, the surface of the casting is usually uneven where it ought to be even. This unevenness, or the irregular swells and hollows on the surface of a casting, is caused by the unequal shrinkage of the iron of different qualities.

Specification Tests of Cast Iron.

Three bars, each 3 ft. 6 in. long, 2 in. deep, and 1 in. wide, to be cast on edge in dry mould from each melting at which any of the specified work is cast. These bars to be tested separately as follows : The lower side, or thin edge, of the casting to be placed downwards upon rigid bearings, with 3 ft. clear span, each bar to deflect not less than $\frac{3}{10}$ in. with a load of 25 cwt. in centre having a bearing not more than 1 in. wide upon the bar, to break with a minimum load of 28 cwt. and an average upon the three bars of not less than 30 cwt. Samples prepared in lathe to bear $2\frac{1}{2}$ tons per square inch tensile strain before loss of elasticity, and to break with not less than 7 tons per square inch, or an average on three samples of $7\frac{1}{2}$ tons. Test bars are sometimes cast as projections from an important casting and broken off for testing, but this is a bad method, and gives 10 to 20 per cent. lower results. No difference is made in testing the material for small or large castings, so far as their size is concerned, but large castings are often required to be stronger in proportion than small ones, and the test may be varied as to its intensity to meet such cases.

Testing Iron Rivets.

Rivet shanks should be capable of being doubled close when cold and also when hot, without showing any sign of fracture. They should also bear a hole punched through them when hot with a round taper punch, and enlarged to diameter of shank without showing signs of cracking or splitting. The heads when hot should bear hammering down to two and a half times diameter of shanks, or to $\frac{1}{8}$ in. thickness without fraying or cracking at the edges. There should be a slight radius instead of a sharp angle at the junction of the head with the shanks.

Distinguishing Iron from Steel Rolled Joists.

The old test for distinguishing between the two substances was to place a drop of dilute nitric acid (1 of acid to 4 of water) on a brightened surface, when, according to Bloxam's "Chemistry," if the metal was steel a dark grey stain would be produced owing to the separation of the carbon, while wrought iron would show a greenish grey stain. The harder the steel the darker the stain, but the mild steel used at present would probably not show much darkening. When the metals are slung in a rope and sounded with a hammer, the steel will give a clearer ring than the wrought iron, but if struck while lying on the ground various local causes may modify the sound. The cut surface of steel looks, to an experienced eye, rather whiter and closer grained than wrought iron, but the difference between the two is more easily shown by chipping off a small shaving, when the steel will be found to be the tougher of the two. The tempering test might be applied to a small piece of metal, but some steel is so mild that this test might not perhaps be quite satisfactory. A fairly reliable test is to try to break off with a hammer a small piece from the edge at one end of the joist. Steel would only bend, and wrought iron would probably show a longitudinal crack, and a piece of the iron might break off. Steel joists of the same over-all dimensions as iron joists are usually about 15 per cent. lighter, because the steel joists are rolled thinner. Some makers roll the word "Steel" as well as their own name on the web, and this may be taken as conclusive. Rolled iron joists are not now made, but old ones are still in use.

Distinguishing English from Foreign Rolled Joists.

Foreign joists are frequently met with in odd sizes, the measurements being reckoned in millimetres instead of inches (or instead of quarter-inches in the width of some of the smaller joists), but the width, etc., of the foreign joists may, in some cases, differ so little from English sizes as not to be distinguishable from them. In foreign joists also the web may be abnormally thickened in order to make up the weight, but this extra thickness is valueless, and the joist is therefore considerably weaker than an English joist of the same weight. The larger sizes of foreign joists are, more often than is the case with English joists, made up of portions welded together, as may often be seen when the ends are examined. English stock lengths (6 ft. to 10 ft. up to 30 ft. by 1 ft. or 2 ft. changes) generally have clean sawn ends. Rolled steel joists usually have the maker's name or trade mark rolled on the web in raised characters, besides which joists imported into this country must be marked with the country of origin, as, for example, "Made in Belgium." If only a short piece of joist is available for examination the trade mark may have been cut off, but if the joist is of Belgian origin the edges will be more square than is usual with English sections. Foreign joists are rather apt to be slightly twisted, to have the flanges buckled, and the depth not uniform. If foreign joists are adopted the specification should always contain the following clause: "All joists to be delivered perfectly straight and parallel and free from flaw or defect of any kind." American joists are clean and well made, but are rather apt to be cold short or brittle from excess of phosphorus. Those made by the best makers, such as the Carnegie Steel Co., are, however, reliable and cheap. Other matters to look for when inspecting the erection of work are rivets omitted or fractured, bolts not tightened up or bolts without nuts, bolts not fitting the holes, holes punched instead of drilled, holes too near the edge, etc.

Rust and Corrosion of Steel and Iron.

Steel rusts less than iron because of its greater purity and freedom from scale. Steel rust usually rubs off as fine dust. Corrosion of wrought iron is often greatly due to the galvanic action which sets up between the iron and the small portions of uncombined carbon which are present in the iron. This action does not consume the carbon, but only the iron (iron is said to be electro-positive to carbon); but in mild steel, as there is no uncombined carbon, there is no tendency to this galvanic action. It is, however, necessary thoroughly to remove the mill scale from the steel by pickling in dilute sulphuric acid, as this scale also is capable of setting up a galvanic action with the steel.

Some Terms Explained.

Bearing area of rivets is the diameter of each multiplied by the thickness of plate or plates against which they bear in the same direction.

Bending moment at any point is the algebraic sum of the forces acting on the beam between that point and one abutment multiplied by their leverages to that point.

Camber is an upward curvature given to a girder or beam to avoid the appearance of drooping in the centre.

Channel iron is like a double angle iron or trough, and is used for compression bars in cranes, girders, and roofs.

Compression is the effect produced by a thrust, or pressure, on the ends of a piece in the direction of its length.

Cover plate is a short plate covering a joint in other plates or angle irons to provide sectional area for that lost by the severance at the joint.

Cross section is the shape of the cut surface when a piece is cut across.

Deflection or bending is the transverse displacement of part of a beam due generally to a transverse load, but sometimes to a thrust.

Elastic limit is that point in the straining of a beam up to which it will recover its original shape and dimensions after removal of the load.

Flange is the term used for the upper or lower part of a beam where material is collected to resist the longi linal stresses of tension or compression.

Moment of tance is the strength of a beam to resist bending moment, which it must be equal ; it depends upon the size and shape of t cross section and the stress allowed upon th material.

Permanent set an alteration of shape produced by strain a piece beyond its elastic limit.

Pitch is the di tance from centre to centre of rivets or bolts.

Reaction is the term used to express the resistance of the supports to the pressure produced by a girder.

Shearing stress t any section is the tendency of the material on one side of the section to slide upon the material at the other side of the section.

Strain is the alteration of form produced by the effect of a lo upon a beam.

Stress is the effect produced on the particles of a beam by a load.

Tension is the name given to the effect of a direct pull, or the same result produced by other means.

Thrust is the name for a push, or pressure produced in the length of a piece.

Torsion is the technical term for twisting, one end being fixe while the other is partially rotated.

Web is the part intermediate between the flanges provided to keep them apart and resist the shearing stress s.

Whitworth standard is a certain pitch of thread for bolts and nuts, according to their size, to give the best result, and procure uniformity throughout the kingdom.

BRICKS AND BRICKMAKING.

Desirable Qualities in Bricks.

GENERALLY, bricks should be rectangular and parallel in shape and uniform in size, flat, but not so smooth as to prevent adhesion of mortar, free from stones, and from lumps of lime and cracks, and without twist, warp, bend, or excrescence. They should be dipped in water before laying, to counteract dryness and dust. The length should be equal to twice the width plus the thickness of one mortar joint in order to obtain good bonding and uniform work. The frog must not be too deep, or it will require too much mortar. When made without a frog the shape must be truer to allow of thin mortar joints. For external work the bricks should be sufficiently well burnt to withstand the weather —that is, they should be impervious to rain. The arrises and corners should be sharp and square, and the colour uniform. For footings, they should be burnt to the point of incipient vitrification ; the colour is immaterial, but the shape should be sufficiently regular to permit of their being fairly bedded. For internal work to be covered, they should be sound and regular, but need not be hard burnt if protected from moisture. For internal work exposed, they should be sound, hard, regular, and uniform in colour. For paving they should be extremely hard (that quality being obtained by using a more plastic clay and less sand in the manufacture), and sufficiently regular in shape to bond with each other and make a fairly even surface. For gauged work they should be full size with sharp arrises, without stones, and with sufficient sand in their composition to permit of cutting and rubbing with facility. As regards testing, when two are struck together they should give a more or less metallic ring, which will be very pronounced in the case of hard-burnt bricks of good quality, and dull in the case of soft bricks. Generally the ring of the trowel while the bricklayer is at work will tell the quality of the bricks. If they are to be exposed to the weather, they should not absorb more than one-sixth to one-eighth of their weight when dipped in water after previous drying, or one-fifth if left in water twenty-four hours. The hardest bricks will sometimes absorb as little as one-fifteenth. A good facing brick should resist the knife, and a good rubber should resist the finger nail until the outer skin of the brick is removed. If required for important work where a great load has to be carried, or a new quality or make of brick is proposed to be used, specimens should be submitted to experts for testing the crushing strength of a small cube or a whole brick, and also the crushing strength of a pier built in lime or cement mortar. A good brick cannot be broken by throwing it on the ground, but it can be broken by holding one end and striking the brick about two-thirds along against the edge of another one. The appearance and squareness of the fracture, and the force of blow required, will indicate some of the qualities of the brick. The structure should in all cases be uniform and compact.

Composition of Brick Earth.

Brick earth is a sandy or loamy clay with a small proportion of lime, magnesia, iron, etc. A natural clay is generally deficient in some ingredient essential for the manufacture of good bricks, and hence varieties of clay are mixed together, or the missing ingredients are added. In their natural state, brick earths may be classified as: (1) Plastic, pure, or strong clays, called by brickmakers "foul" clays, composed of silica and alumina, with very small proportions of lime, magnesia, soda, and other salts. They shrink and warp too much if used alone, and are very hard when burnt. (2) Loams, sandy or mild clays, consisting of clay (silicate of alumina) and sand (free silica). The sand prevents the brick from shrinking, warping, and cracking, and reduces its hardness, but loams generally require the addition of lime as a flux to bind the materials together, and washing to free them from pebbles. (3) Marls or calcareous clays containing a large proportion of carbonate of lime (chalk). A good brick earth, natural or artificial, should contain such proportions of silica, alumina, carbonate of lime, carbonate of magnesia, oxide of iron, etc., as to form a sufficient flux to fuse the constituents at a furnace heat, but not so much as to cause them to vitrify so as to run together or destroy their regular shape. London brick earth contains by analysis about $\frac{2}{3}$ silica, $\frac{1}{4}$ alumina, $\frac{1}{100}$ oxide of iron, $\frac{1}{200}$ carbonate of lime, and remainder organic matter.

Colour of Bricks.

The colour of bricks depends upon the composition of the clay, the kind of sand used for moulding, the dryness of the bricks before burning, the temperature at which they are burnt, and the amount of air admitted while burning. Pure clay, free from iron, burns white, but the white colour is generally obtained by adding chalk to the clay. Iron in small quantities produces a light yellow, increasing to orange and red in proportion to the quantity. When it approaches 10 per cent. an intense heat will convert the red oxide into black oxide, which, in combination with the silica, produces a blue or purple colour. Manganese added to iron darkens the colour still more. Lime in presence of iron produces a cream colour, and in larger quantity brown. Magnesia in presence of iron makes the brick yellow. London bricks are naturally pinkish, but the use of Woolwich sand in moulding causes them to burn with a pale buff or warm yellow tint.

Materials used in Brickmaking.

Alumina is an essential and the chief constituent in brick-earth, and supplies the tough elasticity of the brick, but it will not fuse properly without a small portion of iron or lime to assist its union with the silica or sand. If an insufficiency of sand is present the brick will tend to warp and be too hard.

Clay is a hydrated silicate of alumina, with a small proportion of lime, magnesia, iron, etc. The principal component is alumina, or oxide of aluminium (Al_2O_3), and this has with it both combined and free silica (SiO_2) in varying proportions—say $Al_2O_32SiO_2$.

Kiln-burnt clay is partially vitrified and all the water is driven off, the material being so altered in character as to be incapable of solution in water. This is the theory of brickmaking.

Lime in a finely divided state assists in the partial vitrification which is necessary for good bricks, acting as a flux. It causes incipient vitrification in the burning, which renders the brick harder and more durable. In excess it causes the bricks to run and become misshapen. When in large pieces it causes the bricks to "flush" and burst after being laid, by slaking and swelling when moisture reaches it. In visible particles it is highly injurious, as it becomes a hydrate on exposure to moisture after burning, swelling and bursting off small portions of the surface or skin.

Salt is injurious if present in more than very small quantities, as it acts as a violent flux, causing the bricks to run together before they are properly burnt. Being hygroscopic, it attracts moisture to the walls when in position.

Sand in moderate quantity prevents excessive shrinking in burning and also cracking while drying. In excess it makes the brick brittle.

Sun-dried clay is the condition of bricks upon the removal from the hacks, where the clay merely loses some of its moisture, but is unaltered in character, and can be again made plastic.

Processes of Brickmaking.

Sand moulding.—By this method the mould is dipped in or sprinkled with dry sand, breeze, or fine ashes before the clay is pressed into it, in order to prevent the adhesion of the clay to the mould. Bricks made in this way are more shapely and with sharper arrises than by slop moulding; they have a better surface, which is often modified in colour by the sand, and the sand sometimes vitrifies on the surface, producing a semi-glaze; but if it does not, the bricks must be well wetted when laid, or the mortar will not adhere.

Slop moulding.—By this method the mould is dipped in water each time of using, to prevent the clay sticking to it. Surface cracks frequently occur, and the bricks do not keep their shape so well.

Drying bricks.—*Bricks* are dried before burning to allow of a gradual shrinkage without cracking. It is necessary even to protect them from a very drying wind or from the direct rays of the sun when they are freshly laid on the hacks, to prevent them from cracking and warping.

Close-clamping.—This is the process used when the bricks are made with a due proportion of breeze (sifted dustbin refuse) incorporated with the brick-earth in a pug-mill, which renders each brick a fire-ball—that is, containing within itself the material necessary for its vitrification. It is the usual method adopted in the neighbourhood of London. In building these clamps, the ground is levelled and slightly hollowed, two 9-in. battering walls of burnt brick are built about 45 ft. apart, burnt bricks are laid down as paving, then two courses of burnt bricks on edge, one laid diagonally and the other square, with 2-in. spaces between (called "scintling"), and a trough or live hole 9 in. wide along the middle, in which faggots are laid and all the spaces filled up with breeze. Upon this the dried raw bricks are laid close together (called "close-bolting") in alternate layers of all headers and all stretchers, as follows:—An "upright" or double battering wall is built through the centre, and a number of walls or "necks" on each side leaning towards it, but all courses horizontal, except towards the ends of the uprights and necks, where a tilt is given to prevent the bricks falling outwards in burning. Above the first course of raw bricks a 7-in. course of breeze is placed, over the second course 4-in., over the third course 2-in., and over the remainder either none at all or $\frac{3}{8}$-in., according to the composition of the bricks; while the top course has 3 in. of breeze over it and a covering of burnt bricks. The outsides are then covered with burnt bricks or plastered with clay. In clamp-burnt bricks traces of the breeze mixed with the clay can generally be seen; they are often slightly irregular in shape, and warped.

Open-clamping.—This is the method of burning bricks which do not contain their own fuel, by building into clamps, and arranging them in open order the same as in a kiln, but with the fuel interspersed and the whole covered in with a temporary casing of old burnt bricks. This method is not much practised.

Kiln burning.—This is the method usually adopted in permanent brickfields away from London. The kiln is generally a strong rectangular open-topped brick building, having large doorways at each end, and openings or fire holes along the sides, and may be described as a furnace in which the bricks are burnt without coming into contact with the fuel. The bricks are placed diagonally with spaces between, and reversed in adjacent courses, so that the heat may pass round them. Common bricks are arranged at the bottom, and, to form flues to diffuse the heat from the fire holes, best bricks and tiles are placed in the central spaces, and common bricks again at the top. The crossing of the bricks gives a "brindling" to them, or stripes and marks of varying colour, owing to the partial exposure to the heat; and this is particularly seen in Nottingham bricks. When uniformity of colour is required, care has to be taken to place the bricks with the faces over each other. Kiln-burnt bricks have often light and dark stripes on their sides caused by being arranged for burning with intervals between them. Where the brick is exposed it is burnt to a light colour; where it rests upon or against other bricks it is dark. In some cases care is taken to prevent this, and the best kiln-burnt bricks are of uniform colour.

How Bricks are made in London.

In the neighbourhood of London the bricks are hand-made from a yellow clay overlying the London blue clay. The turf is first stripped and sold retail at 8s. per 100 pieces 3 ft. × 1 ft., or wholesale at a lower rate. The vegetable earth is then either sold or wheeled into heaps for use in the gardens which are subsequently attached to the houses laid out on the cleared site. The clay is next dug and wheeled into a heap, together with roughly sifted old dustbin refuse, made up with the addition of coke breeze. A good heap is generally left through the hardest part of the winter, and directly the principal frosts are over the brickmakers begin passing the mixture through a pug-mill, worked by a horse, to prepare it for moulding (see under Malms, p. 30). The moulding is done by a man standing at a small bench, who dabs a lump of clay into an open mould resting on a stockboard to form the kick or frog. The mould has been previously dipped in water to prevent the clay sticking, and as soon as the clay is pressed home the moulder strikes off the surplus and throws the moist brick out on to a pallet at his side, when it is taken by a hack boy with another pallet and placed with other bricks on a long barrow. When the barrow is full it is run along wheeling plates of wrought iron, about 15 ft. long, 12 in. wide, and ¼ in. thick, to the hacks. Here the bricks are laid upon a slightly raised foundation, mostly in diagonal courses with spaces between, and protected in wet or hot sunny weather by small light portable roofs. In the course of a month or six weeks, when the bricks are dry, they are built into a clamp and fired. The site for the clamp having been prepared by levelling and draining, a foundation of burnt bricks is laid, close, and on edge; then another course on edge, but in parallel lines diagonally, with spaces of 1½ in. or 2 in. filled with breeze, then another close course of burnt bricks is laid. Then a covering of breeze 6 in. thick, and upon this the first course of raw bricks is laid. Then another layer of breeze, and two or three courses of raw bricks. Thin layers of breeze are laid at intervals between every few courses of raw bricks, and the top and sides are covered by a layer of place bricks, smeared with clay or covered with boards on any side where the wind is too strong. Live holes are left at the bottom, and fire channels filled with faggots and breeze, so that the clamp may be thoroughly ignited. In about a month's time the bricks are ready for being sorted out and stacked for sale according to quality, the site being utilised for another clamp, and so on until the clay is exhausted.

Varieties and Uses of Bricks.

Bats are broken bricks, and when cut to sizes are used for completing the bond in brickwork.

Blue Staffords are made from various clays and marls containing 7 to 10 per cent. oxide of iron, burnt at a very high temperature to convert the iron into protoxide. They are very strong, durable, non-absorptive, and unaffected by the weather. Used for important engineering works as plinths, quoins, copings, paving, etc. Colour, deep blue-black throughout when properly burnt, and glazed surface. Imitation blue Staffords have only a surface wash of iron to give outside colour, and are red inside.

Burrs and Clinkers are masses of brickwork fused together, or single bricks overburnt or partially vitrified. Generally found near the "live holes" of a clamp. Used for building rustic boundary walls and artificial rockwork.

Brindled bricks are so called from the light and dark stripes produced on their surfaces by partial contact with other bricks in burning. They are usually hard machine-made bricks of a white or gault clay, showing a red and white mottled appearance when burnt. They are used for rough work, such as manholes, sewers, and warehouses, and would be set either in lime or cement mortar.

Clamp-burnt bricks are those burnt in stacks with spaces or flues left at certain intervals throughout the stack. Layers of coke breeze or dustbin ashes are placed near the bottom, and flues are formed leading from the live holes or eyes in which the fire is first started.

Dinas bricks—see page 29 under Firebricks.

Dust bricks are blue bricks, in the moulding of which coal dust is used instead of sand. They have glossy surfaces, are very hard, and are used for paving.

Dutch clinkers are very hard, well-burnt bricks vitrified throughout, occasionally warped in burning. They are much smaller than ordinary bricks, and measure about 6 in. × 3 in. × 1½ in. Generally bright buff in colour, but the addition of oxide of iron will make them black. Used for paving only.

Exposed brickwork should be built with bricks least affected by the weather, such as Staffordshire blue bricks. For the particular purpose named, the bricks should be non-absorptive; that is, should not absorb more than 3 per cent. to 4 per cent. of their weight of water when immersed for a considerable time The other qualities of the best Staffordshire blue bricks would also be suitable, namely hardness, weight, strength, and regularity of shape.

Fareham red bricks are made from a moderately plastic clay which is found in very deep beds around the town of Fareham, and in other places in the neighbourhood. They are dressed or batted when partially dry, and thus brought to a very true surface. They are also carefully burnt in small oven kilns holding from 20,000 to 30,000 each. These bricks are of a fine deep red colour, and have been much used in London for superior buildings. Sometimes these bricks are rubbed so as to obtain very fine surfaces and thin mortar joints, but this removal of the outer skin is bad, as it tends to make the brick decay quickly under atmospheric influences. Size, building bricks, $8\frac{1}{2}$ in. by $4\frac{3}{16}$ in. by $2\frac{5}{8}$ in.; rubbers, $10\frac{7}{8}$ in. by $4\frac{7}{8}$ in. by $2\frac{7}{8}$ in.

Firebricks are those which resist the action of fire more than ordinary bricks; that is, do not contain the materials for forming a flux, and do not readily melt. The best are the Dinas bricks of Glamorganshire, composed of a so-called fireclay containing 97·6 per cent. of silica and therefore being practically sand only. About 1 per cent. of lime is added, with sufficient water to enable the bricks to be pressed in a mould before being dried and burnt. The bricks are moulded by machinery and kiln burnt. They are very tender, but stand the greatest heat, and are therefore used for the best furnace work The fractured surface shows coarse, irregular white particles of quartz surrounded by small quantity of light brownish yellow matter. Common bricks contain only about 50 per cent. silica, the remainder being chiefly alumina, lime, and iron.

Gault bricks are made from a band of bluish tenacious clay which lies between the Upper and Lower Greensand formations. This clay, in its natural state, contains sufficient chalk to flux the mass and to give the brick a white colour. The bricks made from this clay are of very good quality, extremely hard throughout, very durable, but difficult to cut. They are stronger than ordinary stocks, and are used for facings. They are generally white, but the lower qualities have a pink tinge, caused by irregularities in burning. Bricks made from Gault clay are generally very heavy; to remedy this, a large frog is sometimes formed in the brick, or it is perforated throughout its thickness. Bricks of this description are manufactured at Burham near Rochester by the Burham Co., at Aylesford near Maidstone, and in the neighbourhood of Folkestone, Hitchin, and other places. Suffolk white bricks are made from Gault clay with a large proportion of sand, which makes them useful for rubbers. Size, $8\frac{5}{8}$ in. to 9 in. by 4 in. to $4\frac{3}{8}$ in. by $2\frac{5}{8}$ in.

General Walling is best built with good London stocks.

Glazed bricks are best when manufactured from fireclay, and have usually a white glazed surface produced by a thin coating of china clay over the face, which becomes vitrified in burning, but the colour may be varied if desired by the addition of colouring matter to the wash. They should be set in Portland cement mortar, and may be flat-pointed in Keene's cement. They are used for facing walls of larders, basement areas, sanitary disconnecting chambers, and in various cases either for reflecting light or permitting easy cleaning.

Good bricks when struck against each other should have a clear ring. When broken, the fractured surface should show incipient vitrification all through the brick. They should be free from flint pebbles and lumps of lime, regular in shape, with arrises square and sharp. Rubbers and cutters should not be soft enough to be marked by the finger nail, and stock bricks should resist the scratch of a knife. The surfaces should be fairly rough, although regular, so that the mortar may adhere thoroughly and make a thin joint.

Grey stock bricks are good ordinary hard-burnt London-made bricks, slightly irregular in colour, and not quite perfect in shape, used for the general mass of ordinary brick walling.

Grizzles and Place bricks are soft and under-burnt; used in partitions.

Heavy foundations are best built with best hard London, stocks or blue Staffordshire bricks.

Kiln-burnt bricks are those burnt inside a rough open-topped brick building, with thick walls and fire-holes at the bottom along each side. The difference between clamp and kiln-burnt bricks is that the former have coke breeze in their composition and are very irregular in colour and hardness, some being so overburnt as to be useless, and others from being insufficiently burnt have to go into the next clamp to form the foundation. Kiln-burnt bricks are free from ashes, more uniform in colour and shape, and, as mentioned under "Kiln Burning," on p. 27, are often brindled or marked in stripes where other bricks have lain across them. The reduced waste on kiln-burnt bricks compensates for the extra cost of building the kiln.

King closer is the piece left after cutting off the corner of a brick $2\frac{1}{4}$ in. one way and $4\frac{1}{2}$ in. the other, as in Fig. 38. It shows as a closer on face, and bonds in better with the remaining bricks. Used in the jambs of window and door openings, where there is a recess at the back of a half-brick reveal, to avoid straight joints in the thickness of the wall.

Machine-made bricks may generally be easily distinguished, if wire-cut, by marks of the wires and the absence of frogs; if moulded, by the peculiar form of the mould, letters on the surface, etc., and sometimes by having a frog on both sides, or a number of holes pierced through to lighten the brick. In many cases the marks made by the pronged forks used for hacking the bricks may be seen on their sides.

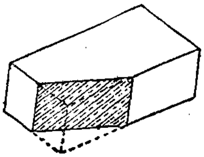


Fig. 38.—King Closer.

Malms are the best clamp-burnt building bricks, usually sorted into several different qualities. They are so called because made from artificial malm, representing the natural marl or calcareous clay, and composed of clay with about 15 per cent. of cinders, and 6 per cent. of chalk all ground together and run out as slurry, and hand moulded as soon as sufficient moisture has evaporated. The better qualities are of uniform colour and regular shape, and are much used as facing bricks. Malm bricks are made with a mixture of clay and chalk prepared in the following manner:—The clay is dug in the autumn, and at once tipped, together with a proportion of ground chalk in pulp, into a wash mill. This consists of a brick-lined circular tank in which are revolving harrows, knives, or implements of some kind to disintegrate and mix up the clay and chalk. The exact proportion of the chalk differs according to the composition of the clay, but in some cases the chalk is about one-sixteenth of the bulk of the clay. The mixture, having been reduced to a creamy consistence, is strained off through fine gratings into large shallow tanks, and there left till it is nearly solid. In some places the preparation of malm is known as washing. After that it is soiled in layers from 1 ft. to 3 ft. deep, that is, covered with about one-sixth its bulk of screened cinders, and allowed to remain during the winter. In the spring the hacks are dug out, the layers of clay and ashes being thoroughly incorporated in a pug-mill, and the material is formed into bricks by hand moulding. The bricks are then stacked in a drying shed or on hacks, and afterwards they are carefully burnt to keep the shape and colour good. They are a yellowish buff colour, soft and uniform in texture. "Best" malms are used for gauged arches, moulded quoins, and general rubbed work; "seconds" for best facing bricks, and "pickings" for inferior work. Malms are the best class of bricks for ordinary building; they are divided according to their condition after burning, and are sorted out under different names according to the composition and appearance, thus:—*Malm Cutters.*—For gauged arches and other rubbed work. *Best Malms.*—For facing in best work; uniform yellow colour. *Malm Seconds.*—Similar, but not quite equal in quality. *Malm Paviors.*—Sound hard building bricks of browner colour. *Malm Pickings* are the rejected bricks from the better qualities.

Place bricks or grizzles are generally reddish in colour; also known as samel bricks. They are underburnt bricks, generally from near the outside of the clamp or kiln, always soft inside, and sometimes outside also, are very liable to decay and unfit for good work; they are, however, often used for the inside of walls, and chiefly for brick-nogged partitions. They are very bad weathering bricks, and may be seen crumbling away in many garden walls. When used for exterior work, they should be rendered in cement to protect them from decay.

Pressed bricks are made by placing raw bricks, when nearly dry, in a metal mould or die and subjecting them to considerable pressure in a press. This consolidates the materials, and causes the bricks to have smooth faces and sharp arises. They are then usually burnt in a kiln. Used in heavy engineering work and foundations. Would be good for facings and gauged work, but the faces are apt to scale.

Purpose-made bricks are those which are specially moulded to shapes suited for particular situations, such, for example, as the voussoirs of arches struck to a quick curve, the corners of obtuse-angled structures, etc. There are several advantages in having the bricks thus purpose-moulded, as cutting is saved, and the surface-skin of the brick is left intact, which enables the brick to resist the weather far better than if the surface were removed by cutting.

Queen closer is the half of a brick cut lengthways, as in Fig. 39; theoretically 9 in. by 2¼ in. by 3 in., but, practically, in two parts, each 4½ in. by 2¼ in. by 3 in., including the thickness of the joints. Used for making up the bond in piers, quoins, and ends of walls.

Red facings are built with best Leicester pressed red facings for ordinary work; Fareham red bricks for better work, when deep red colour is desired.

Rubbed and gauged yellow arches are best built with Suffolk malms or malm cutters.

Shippers is the name given to stock bricks of a somewhat inferior quality, being supposed to be taken by ships as ballast. They are better than common stocks, but not equal to paviors. Used for facing and for 9-in. walls.

Stocks are hard-burnt bricks, fairly sound, but more blemished than shippers. They are used for the principal mass of ordinary good work. Hard stocks are overburnt bricks, sound, but considerably blemished both in form and colour. They are used for ordinary pavings, for footings, and in the body of thick walls. Stocks may be malms, washed, or common stocks, according to the manner in which the earth for them is prepared. In the better qualities they are made somewhat as described for malms (p. 30), but the common qualities consist merely of brick earth tempered in a pug-mill with the addition of coke breeze, and the bricks are burnt in a clamp. They are divided according to their condition into *Washed Stocks.*—The commonest malms, used for ordinary building. *Grey Stocks.*—Good bricks, but irregular in colour; used for railway work. *Rough Stocks.*—Rough in shape and colour; often used in foundations.

Shuffs are unsound bricks, full of shakes.

Wire-cut bricks are generally machine-made bricks, but may be worked from a handmill The clay being mixed or tempered in mill, is forced out of a rectangular opening, 9 in. by 4½ in., in a solid stream, which is cut by wires at regular intervals of 3 in. to form the separate bricks. The chief objection to wire-cut bricks is the absence of a frog or key on one face for bonding, but they usually have sharp arrises, and are suitable for common facings.

White facings are best built of Gault bricks or Beart's patent No. 1 best selected white facing bricks, made at Arlesey, near Hitchin.

Measurement of Brickwork.

The size of a brick depends on the size of the mould, the nature of the brick earth, and the temperature of burning. Midland and northern bricks are generally larger than London bricks and the bricks of the home and southern counties. In the latter districts bricks are usually about 8⅞ in. by 4⅜ in. by 2⅝ in. for stock bricks, and facing bricks may be obtained up to 3 in. thick. The size of a brick depends somewhat upon the nature of the clay and the quantity of sand used. A rod of brickwork in mortar at four courses to the foot and built solid takes 4,352 bricks, and laid dry 5,370 bricks, but allowing for cavities, etc. throughout a building, the outside measurement only requires on the average 4,300 stocks to the rod. The joints in ordinary work will be at least ¼ in. thick with London stocks; but the bricks vary slightly in size and are not perfectly true, so that in some cases the joints will be rather more than ¼-in. thick By the standards named above, a 1½ brick wall would measure 13¼ in. thick, but it is usually 13½ in. thick, owing to a slightly larger joint being left in the thickness of the wall to permit of fair work on the face, with bricks of varying size. Bricks differ by greater amounts in the length than any other dimension, owing to the varying shrinkage produced by the burning, but the same percentage of difference will occur in each dimension from this cause. Although nominally called a 14-in. wall, it does not often reach beyond 13½ in. in thickness. The London standard for finished brickwork is four courses to the foot. The standard rule for thickness of joints is that four courses of the finished brickwork shall reach 1 in. higher than the same number of bricks laid dry. With Portland cement mortar the joints may be thicker than with lime mortar.

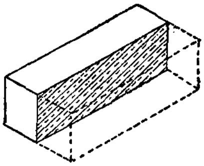


Fig. 39.—Queen Closer.

FOUNDATIONS.

The Term Defined.

The term "foundations" in buildings applies to those parts of a structure which are below the base of the walls, and the term also serves to describe the soil supporting them. The wall itself is the only part essential to the superstructure as such, and the footings, etc., are only necessary to provide a support for the walls, and for that reason they are classed with the concrete, gravel, etc., upon which they rest.

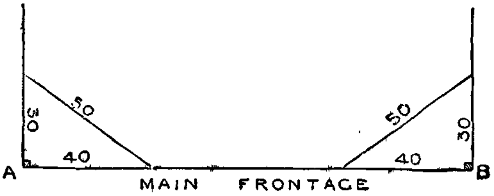


Fig. 40.—Squaring Lines in Setting Out.

Setting Out Foundations.

The general principles of setting out foundations are: (*a*) Fix the main frontage line according to the conditions of the case; (*b*) set up a square line at each end by measuring off 30 ft., 40 ft., and 50 ft. (or any measurements in the proportions of 3, 4, and 5), as shown in Fig. 40, produce the sides as far as necessary, and if both sides are the same length check the squaring by measuring the distance between the ends, which should equal A B. Whipcord, like that used for a chalk line, will do for the lines, but fine brass wire is sometimes used. The pegs at A and B may be in line with the outside of the concrete. The width of the trench for concrete may be set off at various points by measuring from the main lines, and the pegs may be used for measuring from to get the line of brickwork. Generally, any single wall or pier is set out by lines strained from pegs outside the area as shown in Fig. 41. The lines should always run beyond the net limits, as, for instance, Fig. 42, representing the lines for a rectangular pier, and Fig. 43, showing wall trenches. In general building the first thing will be to sight through to obtain the frontage line, then to range a line at right angles for the flank wall, and afterwards to put in the other lines parallel with the lines already obtained.

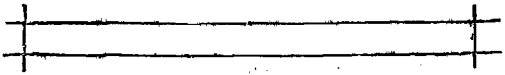


Fig. 41.—Setting Out Single Wall.

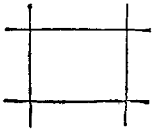


Fig. 42.—Setting Out Pier.

Squaring Lines in Setting Out.

Instead of setting out a triangle as above, a framed wooden square having 6 ft. or 10 ft. sides, and a 45° or 60° brace to hold the sides together, may be used. Such a square is found in various departments on a building. Or a cross staff, or an optical square, may be used. In any rectangular outline such as Fig. 43 a chec upon the accuracy, beyond seeing that tl

Fig. 43.—Setting Out Wall Trenches.

opposite sides are equal, is to see that the two diagonals are also equal. If long lines are to be set out at right angles, a cross staff may be used continuously round the four sides, and probably two or three trials will be necessary before a true square or rectangle can be obtained.

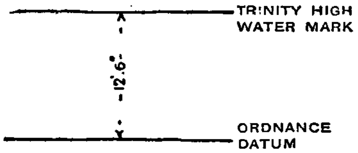


Fig. 44.—Diagram showing Level of Trinity High Water and Ordnance Datum.

Setting Out with Theodolite and Rods.

The choice of method largely depends on the size of the building and the nature of the ground. In important work a theodolite is used, as this will give any required angle absolutely correct; but it is an expensive instrument, and for its use requires considerable skill. For obtaining correct lengths for walls, etc., 10-ft. rods are used, and for the position of piers, windows, etc., long rods are used with the positions marked on. The various details are set off with the ordinary 5-ft. rods, brass tipped, and painted black, with white figures, the section being about 1 in. by $\frac{3}{16}$ in.; or by 10-ft. rods 1½ in. square, painted similarly; or by plain 15-ft. or 20-ft. laths (say from 2 in. by ¾ in. to 3 in. by 1 in.) upon which distances are marked in pencil, with a description of what the marks refer to. Rods longer than 20 ft. are not usual.

Setting Out House Connections in Drainage Works.

The setting out of drainage works is a subject that divides itself naturally into two portions, (*a*) house connections and (*b*) main sewers. In the first group the work is of a very simple character, and a common spirit-level is sufficient to determine the falls. It should be used on a 10-ft. straightedge, tapering from, say, 3 in. deep at the ends to 6 in. deep for a length of 2 ft. in the centre. By driving in small pegs to rest the ends on, the available total fall may be easily ascertained, and then the trench may be got out to a regular gradient. Pipes of 4 in. diameter are the best size for ordinary house drains, and these require a fall of 1 in 40 to 1 in 48, if it can be obtained, that is, ½ in. for each stoneware pipe 2 ft. long. A ¼ in. extra fall may with advantage be given to branch pipes and bends to overcome friction, but it is not absolutely necessary. The sockets should be laid to meet the flow. A 6 in. bed of concrete should be laid for the pipes to rest on, and pockets be left or formed under the sockets to give room for making the joint. The connection to the sewer should be by a junction block two-thirds of the height from the invert, and in the direction of the flow. If the available fall is less than 1 in 96, a flushing tank ought to be provided, and between 1 in 48 and 1 in 96 the bath waste should be arranged to scour the drain on the upper side of the soil-pipe branch.

Setting Out Main Sewers.

In building main sewers it is necessary to have a plan and sections upon which the levels are marked after a survey has been made with this object. The height of the invert of the sewer at the two ends is marked in feet and decimals above Ordnance datum, that is, above the mean half-tide level of the sea at Liverpool, which is 12 ft. 6 in. below Trinity high-water mark in London (see Fig. 44). A sight rail (Fig. 45) is required at each end, and, in longer pieces of sewer, at intermediate points also. This consists of a cross piece nailed at the required height upon uprights set at each side of the trench, generally in an inverted

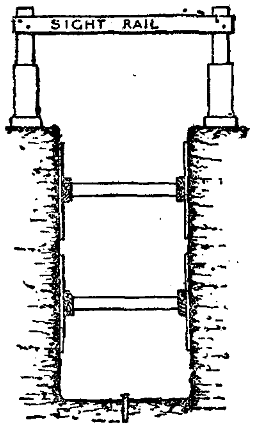


Fig. 45.

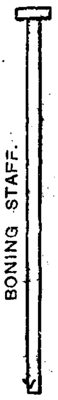


Fig. 46.

Fig. 45.—Cross Section of Trench with Sight Rail.
Fig. 46.—Boning Staff.

drain pipe. A boning staff (Fig. 46), very much like a long **T**-square, is also required; it must be long enough to reach from the sight rail to the invert, say 12 ft. The height of the nearest Ordnance bench mark must be ascertained from an Ordnance map, and the place found on the building or wall, as the case may be, where it will be seen carefully cut in as a crow's foot or "broad arrow." The centre of the horizontal line of the cut shows the exact level given by the figures printed on the map, and this height must be transferred by levelling with a dumpy level to the place where the sewer is to be laid. The height of the sight rail must be set so that the reading taken by a level and staff equals the height of the invert above datum plus the length of boning rod. Then, to obtain the level of invert at any intermediate point, the boning staff is held upright, with the top edge of the T-head sighting truly with the sight rails, and the foot of it is then the exact level of the sewer invert at that spot, a peg being driven to mark the height. Generally the peg is driven down until the boning staff sights through with the rails.

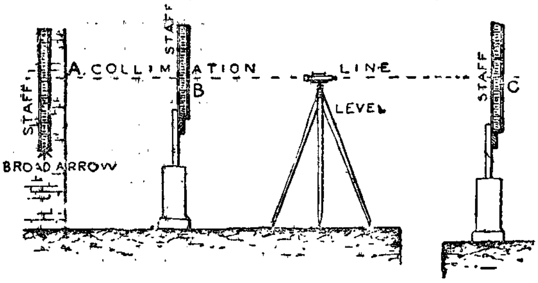


Fig. 47.—Diagram showing Collimation Line, etc.

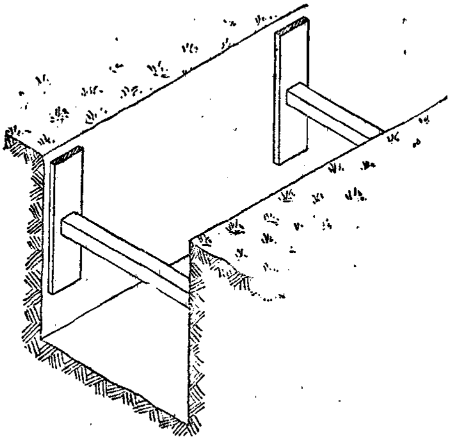


Fig. 48.—Single Poling Boards and Struts in Firm Ground.

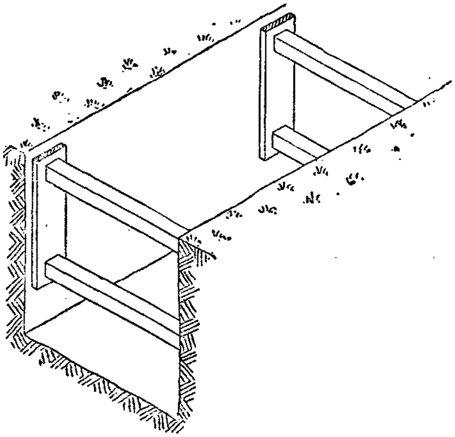


Fig. 49.—Single Poling Boards and Double Struts.

An Example of Levelling.

For example, suppose the Ordnance bench mark at A (Fig. 47) to be 159·70 above datum, and the reading on the staff at that point to be 2·67, the invert under point B to be 150 ft. above datum, and the invert below C to be 147·5 ft. above datum. Then the collimation line will be 159·70 + 2·67 = 162·37 above datum. The reading on the staff at B should

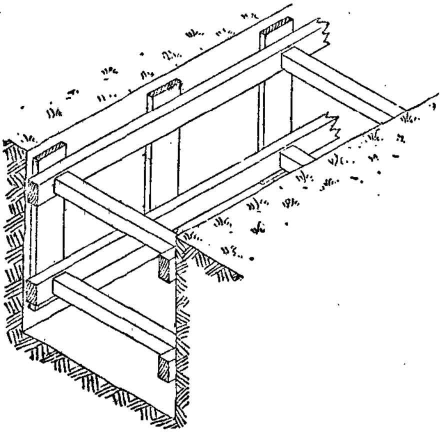


Fig. 50.—Open Poling Boards, Walings, etc.

be 162·37 – 12·0 length of boning rod, minus 150·0 height of invert above datum equals 0·37, and the bottom of staff being marked on the side posts, the sight rail is placed to the mark and nailed. The reading on the staff at c would be 162·37 -- 12·0 – 147·5 = 2·87.

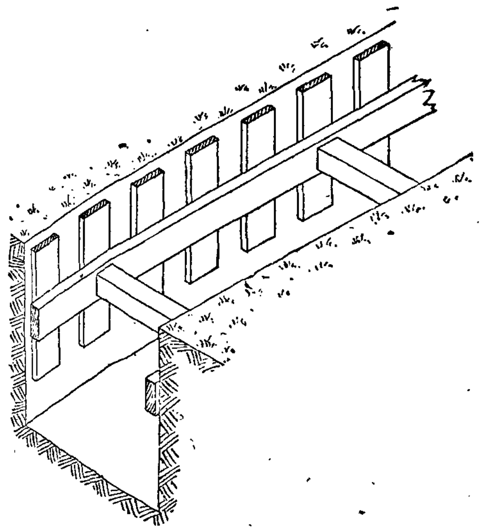


Fig. 51.—Closer Poling Boards with Walings, etc.

Trial Pits.

Before deciding on the kind of foundation to be made for a building, the character of the subsoil should be ascertained. For shallow foundations, trial pits 3 ft. square may be sunk with pick and shovel at various points where the excavation can afterwards be utilised for putting concrete in to support the [illegible] say at the principal angles, and the ground can be pricked with a bar below to see that it is firm underneath.

Trial Borings.

For deep foundations trial pits would be [illegible] expensive, and trial borings are made [illegible] in. to 6 in. diameter—usually 3½ in. The principal tool used is the auger, a hollow cylindrical body with a cutting lip at its lower edge to guide the material into the upper part and hold it there. As the auger brings up all it can get, the operation is called "misering," and the auger is sometimes called a "miser." Jumpers or plain wide chisels of various patterns are used for breaking up any hard material so that it can be raised by the auger, and water is poured down the hole to facilitate the breaking up, particularly if clayey. The strata penetrated must be judged partly by the difficulty of passing through and partly by the samples brought up, allowing for the water which has been poured down. For this reason as little water as possible should be used, so that the natural condition of the subsoil may be more apparent. To prevent the sides from falling in and the bore from choking in loose soil the bore holes sometimes have to be lined with wrought-iron tubes, flush outside. The tools are joined by square rods screwed on in lengths, the top length being with an eye to take a lever bar in the case of the auger, and with lugs to take ropes which pass over pulleys in the case of the jumpers, so that they may be lifted and dropped to perform the requisite disintegration. Measurements and records must be made as the work proceeds, so that a scale section may be made of the various strata. In the case of docks and other deep structures, the borings should be supplemented by some geological knowledge of the district, and, if possible, a geological section map should be consulted.

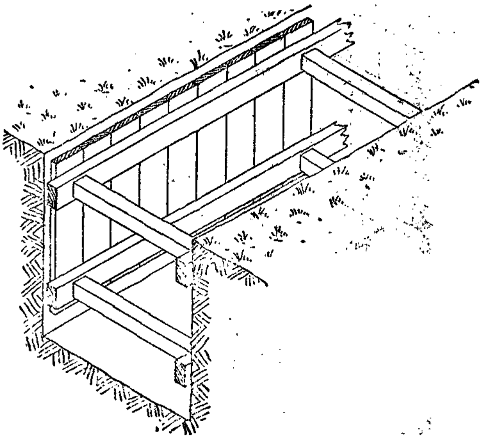


Fig. 52.—Close Poling Boards with Walings, etc.

Timbering for Trenches.

Fig. 48 shows single poling boards and struts in firm ground. Fig. 49, single poling boards and double struts in firm ground. Fig. 50, open poling boards with walings and struts in moderately firm ground. Fig. 51, closer poling boards with walings and struts in loose earth. Fig. 52, close poling boards with walings and

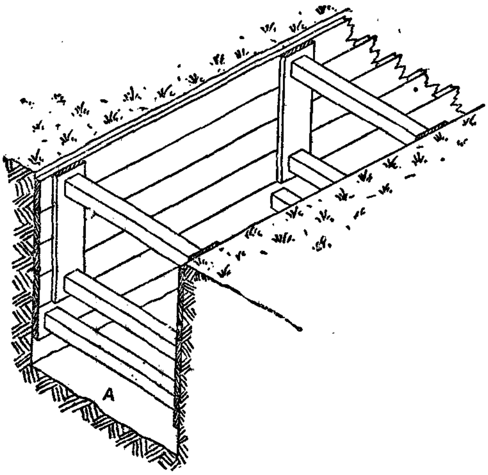


Fig. 53.—Horizontal Poling Boards with Vertical Walings.

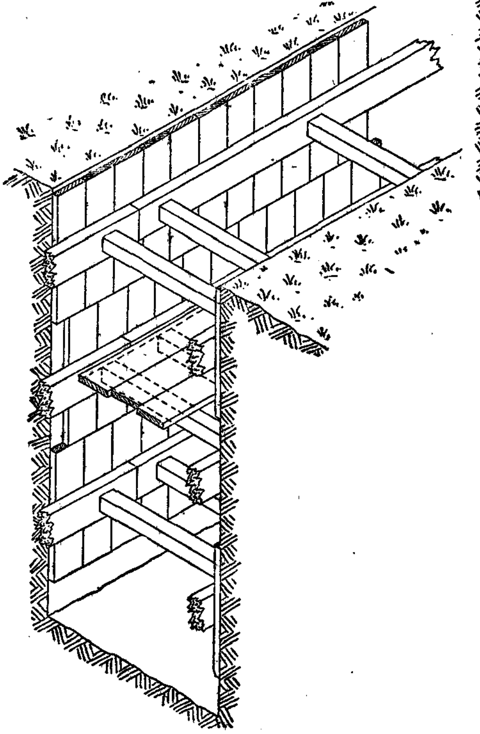


Fig. 54.—Deep Trench with Stage.

struts in loose earth. Fig. 53, horizontal poling boards or sheeting with vertical walings and struts in very loose earth; temporary struts are required to each board as inserted (see A). The trench should taper downwards when the soil is wet to allow of the struts being tightened after the shrinkage due to the removal of the water. Fig. 54, deep trench with stage for throwing out soil. Fig. 55, deep trench in loose soil; the lower tier of poling boards is driven as runners. Fig. 56, pipe lying in deep trench. Poling boards are from 3 ft. to 4 ft. long, and in breadth and thickness from $1\frac{1}{4}$ in. by 7 in. to $1\frac{1}{2}$ in. by 9 in. Walings are 12 to 14 ft. long, and 6 in. by 4 in., or 2 in. by 7 in. to 3 in. by 9 in. Struts are from 4 in. by 4 in. to 6 in. by 6 in., or if round, from 4 in. to 6 in. in diameter; they are placed 6 ft. apart, except at joints in walings.

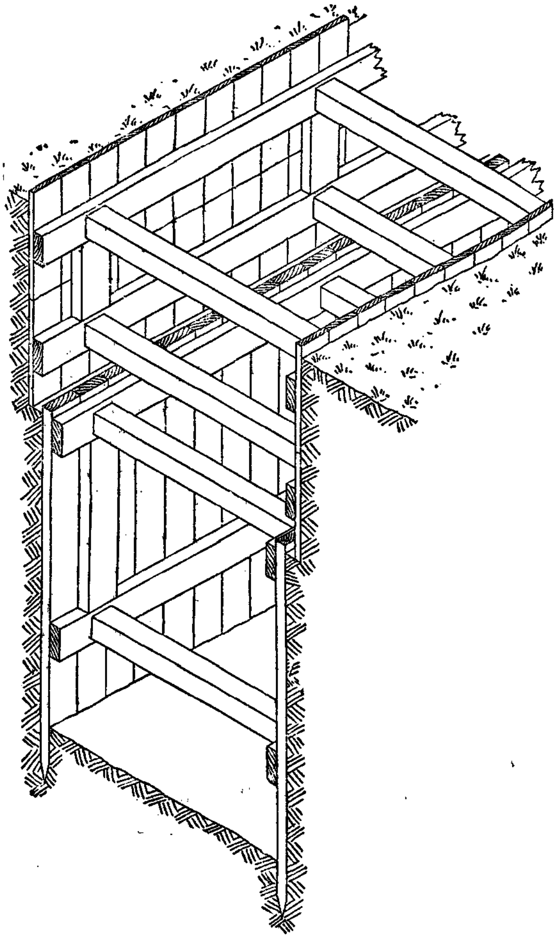


Fig. 55.—Deep Trench with Runners.

Supporting Excavations.

In ordinary loose soil the excavation is made of a sufficient width to carry up the proposed work clear of the timbering, and about 3 ft. deep. For a foundation 4 ft. wide by 10 ft. deep, vertical poling boards 3 ft. to 4 ft. long, and 1¼ in. by 7 in. to 1½ in. by 9 in. in section, are placed on both sides and held up by horizontal walings 12 ft. to 14 ft. long, and 2½ in. by 7 in. to 3 in. by 9 in. in section, kept in place by struts 4 in. by 4 in. to 6 in. by 6 in., or 4½ in. to 6 in. diameter, placed 5 ft. or 6 ft. apart, except at the joints, where they must be closer. A further depth of 3 ft. is then excavated, and another set of poling boards, walings, and struts inserted (see Figs. 54 and 36), and so on for the requisite depth. The lighter struts would be used at the top, and the stronger ones towards the bottom, where the thrust would be greater, or they may all be of the same scantling and put closer together towards the bottom. The ends of the trench are also planked and strutted. At each 6 ft. interval of depth as a maximum, stages must be provided to receive the earth as it is thrown up by the excavators, but the stages may be only from 4½ ft. to 6 ft. apart in height.

Excavations in Loose Soil.

When the soil is looser than in the above example horizontal poling boards are used, as in Fig. 53, so that the excavation need only go down the width of one poling board without support. Temporary struts are then placed at intervals until five or six poling boards are in, when vertical walings are inserted with the permanent struts, and the temporary struts removed.

Runners in very Loose Soil.

In very loose soil vertical poling boards, called "runners," are used, having splayed ends, to cause them to penetrate more easily. As soon as a few inches of the soil is excavated, poling boards are placed at intervals with walings and struts; then the runners are inserted and driven down with mallets or spare struts as the excavation proceeds, so that none of the soil is exposed. The runners may be wedged behind the walings in the intervals of the driving so that they may not slip down. When the first set of runners is fully driven, another set is started inside the previous set, so that if all are driven vertically each additional set slightly reduces the width of the trench, but they are often inclined outwards at the lower ends, so as to maintain a uniform width. Vertical props are used occasionally under the walings where the struts occur, to prevent any accidental slipping, and carry the extra weight of the stages. In running sand, grass or long litter is sometimes used behind the poling boards or

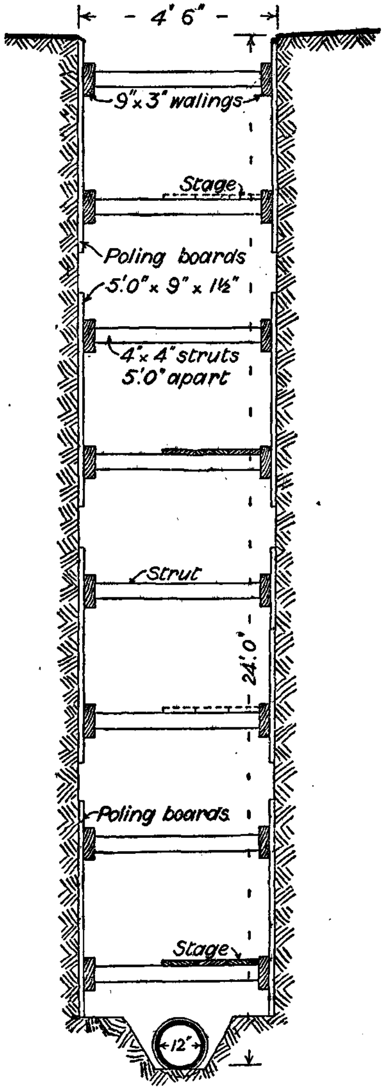


Fig. 56.—Deep Trench for Sewer Pipe.

forced in the joints to prevent the soil from escaping; 3-in. runners, instead of 1¼ in. or 1½ in., may also be used, driven down by a small hand pile engine, and the points kept 5 ft. or 6 ft. below the excavated surface. This is specially useful in quicksand.

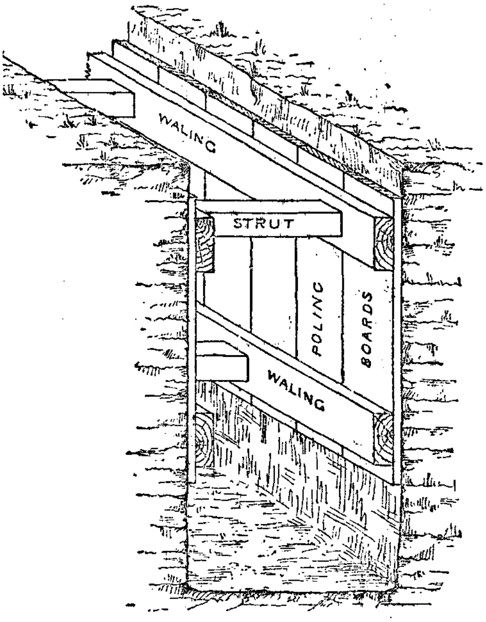


Fig. 57.—Waling, Poling Boards, etc., in Close Timbering.

Removing Timbering.

The timbering is usually removed as the concrete or brickwork and filling proceed; but in some cases, as in a trench for a sewer, the poling boards, and occasionally some of the struts and walings, have to be left in for security. A further explanatory diagram showing the application of these terms is presented by Fig. 51.

Timbering a Large Hole.

The timbering to be used for supporting the soil at the various stages in the excavation of a pit that is to be 18 ft. by 18 ft. by 25 ft. deep is shown by Figs. 58 to 62, it being assumed that the ground is very fair, and will not require close timbering. Fig. 58 is a vertical section; Fig. 59, plan of timbering; Fig. 60, enlarged detail plan of struts and waling; Fig. 61, detail elevation of struts and waling; Fig. 62, wrought-iron dog used in Fig. 61. If the rock at the bottom is sufficiently firm, timbering will not be required in the lower part of the pit. The excavated material is assumed to be lifted in a skip by a crane, and the struts between which the skip travels will need to be securely fixed. The explanation of the terms strut and waling is sufficiently obvious from the illustrations.

Uses of Concrete.

Concrete had comparatively a very limited application before the invention of Portland cement, owing to the fact that lime concrete, which was then generally used, would not set properly in a damp soil; and as all soils are more or less damp, lime concrete had little value beyond that of a simple hard core, or brick rubbish, and was in many cases merely used as a matter of custom. Cement concrete under footings has the advantage of forming a continuous bed, with a definite amount of cohesion that is retained even in a damp situation. On a gravel soil it is not needed, but is nevertheless more often put in than omitted. As an air-tight and water-tight covering over the whole site, with a thickness

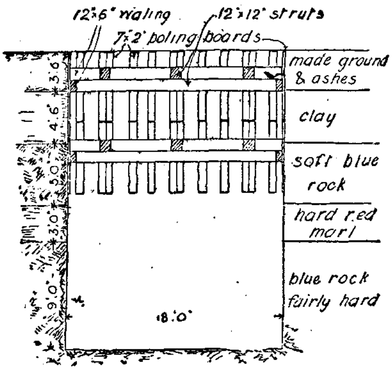


Fig. 58.—Elevation of Timbering a Large Hole.

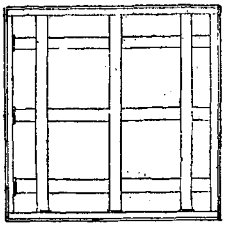


Fig. 59.—Plan of Timbering for Large Hole.

of, say, 6 in., it has a distinct value in excluding ground air and miasma arising from made ground, besides keeping the basement dry. On a clay soil it is necessary either to lay a thicker bed or to strengthen it with embedded iron rods, as otherwise it is liable to be forced up within the inner edge of the footings by the expansion of the clay in wet seasons.

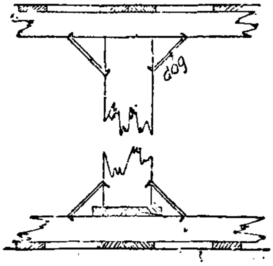


Fig. 60.—Plan of Strut and Waling.

Good Concrete.

To make good concrete, it is requisite first of all to have good fresh cement, properly air-slaked, mixed in suitable proportions with sharp sand and broken stone, clinker, or hard brick, or with pit or river ballast containing sand. If the sand contains any loam it will not make good concrete, as the water necessary in mixing causes the loam to form a muddy coating, and prevents the cement from properly adhering to the hard materials.

Cement for Concrete.

An average specification for the cement would be: The Portland cement to be of British manufacture, from an approved maker. To be properly air-slaked under shelter, in bulk, with a depth not exceeding 2 ft., and to be turned over at frequent intervals. The cement is to weigh not less than 110 lb. per imperial striked bushel, and to pass through a No. 35 S.W.G. sieve of 2,500 meshes to the square inch, leaving a residue not exceeding 10 per cent. Briquettes for testing to have a minimum section of 1½ in. by 1½ in. (= 2¼ sq. in. area) and to be gauged with pure water at 60° F. Every three consecutive briquettes to break at the end of seven days (six in water), with an average tension of at least 850 lb. and a minimum of 750 lb. When gauged into a thin glass tube, the cement must not swell so as to crack the tube, nor shrink so as to become loose. Gauged in a pat ¼ in. thick, on slate or glass, it must bear boiling for half an hour without cracking.

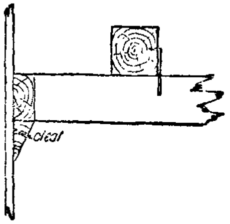


Fig. 61.—Elevation of Struts and Waling.

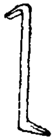


Fig. 62.—Wrought Iron Dog.

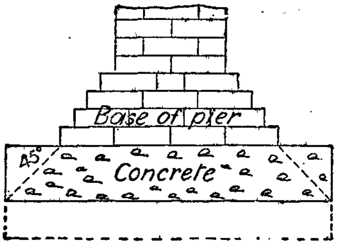


Fig. 63.—Diagram explaining Depth of Concrete under Pier.

Making Concrete.

The concrete for floors should be specified and made as follows: The concrete to be composed of 1 part by measure of Portland cement as described, to 1 part of clean sharp sand and 4 parts of (*a*) coke breeze or (*b*) pumice stone or (*c*) hard brick, free from dust, broken to pass (*a*) ¾ in. (*b*) 1 in. (*c*) 1½ in. ring. The materials to be carefully mixed dry on a wood floor, not more than ½ cub. yd. at one time, and again turned over twice while being watered through a rose. No concrete is to be used or disturbed after being mixed more than one hour; 1 cub. yd. of such concrete requires 27 cub. ft. of broken brick, stone, or shingle, 7 cub. ft. of sand, 7 cub. ft. or 5½ bushels of Portland cement, and 30 gal. of water.

Depth of Concrete Foundations.

The depth of concrete under walls depends chiefly on its projection beyond the footings. In the case of a detached pier, the minimum depth would be given by a line drawn at 45° from the edge of footings, to intersect with a vertical line from the extreme edge of the width, as shown in Fig. 63. The width will be fixed by the area necessary to reduce the pressure on the soil to a safe limit, the minimum being 6 in. beyond the footings all round, which is the width allowed the bricklayer to work in. The concrete is often carried deeper in order to reach a good bottom and to save brickwork. Under a continuous wall the minimum depth will be given by a line drawn at 60°, as in Fig. 64. The reason for the difference is that in a long wall there is more risk of a weak place where the concrete might give way; but of course there is no objection to the angle of 60° being used in both cases. With the pier there is also a larger area of concrete in proportion to the area of brickwork, so that there will be less pressure on the soil with the same amount of projection. These rules are for best cement concrete only.

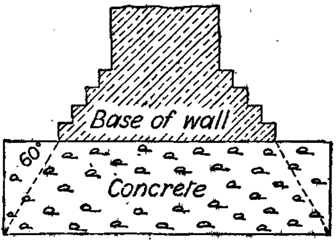


Fig. 64.—Diagram explaining Depth of Concrete under Wall.

Heavy and Light Cements.

Heavy cements, say 115 lb. to 125 lb. per bushel, are as a rule slow-setting, but attain a greater ultimate strength. Light cements, say 108 lb. to 112 lb. per bushel, are quicker setting, but do not reach the same strength. Heavy cements are mostly used in engineering work, and light cements in ordinary building.

Mixing Concrete.

For heavy foundations the concrete used may consist of 1 part cement to 8 parts Thames ballast; for retaining walls, 1 cement, 2 sand, and 6 broken stone, slag, or shingle; for upper floors or roofs, 1 cement to 4 coke breeze, pumice-stone, or clean broken bricks. To mix the concrete, a frame or measure about half a cubic yard capacity, and open top and bottom, should be placed upon a wooden platform and filled up with the ballast. A similar open frame or measure, but one-eighth of the capacity, should be then placed on top of the ballast and the cement filled in, having been previously thoroughly cooled or air-slaked. The small frame should then be lifted off, and afterwards the large one. Two men with shovels, one man on each side, should then carefully mix the materials dry, turning them over to the next platform, where two other men will continue the operation, while a third moistens the mixture with clean water through a fine rose. When thoroughly incorporated and just moist through, the concrete is put into barrows and wheeled to the trench. If the trench is deep the concrete must be tipped down a wooden shoot so as to arrive gently at the bottom, and should then be spread and levelled in layers not exceeding 12 in. thick, the shoot being moved along the trench as required.

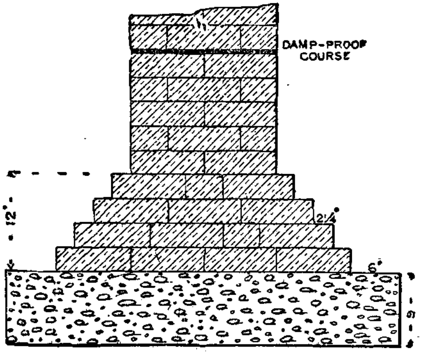


Fig. 65.—Section of Footings and Concrete under Two-brick Wall.

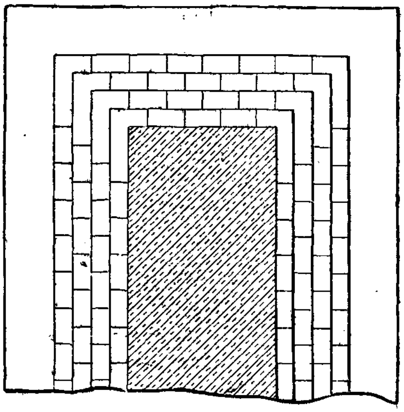


Fig. 66.—Plan of Footings and Concrete under Two-brick Wall.

Rules for Footings of Brick Walls.

The general rules for footings of brick walls are :—(*a*) The number of courses should equal the number of half bricks in the thickness of the wall. (*b*) They should be laid in regular set-offs of 2¼ in. projection on each side. (*c*) The base of the footings should be twice the width of the base of the wall. (*d*) As many headers as possible should be laid in footings, so as to form a good tie between the back and front of the wall, and to render less likely a disturbance in the event of the soil moving. (*e*) Where unequal settlements are anticipated, several strips of hoop-iron bond, properly tarred and sanded, should be laid through the footings. (*f*) When no concrete is used, the bottom course of footings may be double. (*g*) In clay soils concrete is usually laid under the footings, projecting 6 in. on each side and not less than 9 in. deep. (*h*) If it be necessary to spread the concrete wider, in order to reduce the pressure on the earth, the depth should be one and a half times the projection. (*i*) If it be necessary to go down to a considerable depth, as in the foundations for a tower, the footings may project 1⅛ in. in each course, or 2¼ in. with all double courses. The section of a two-brick wall is given by Fig. 65, and a plan of the footings by Fig. 66. The latter also shows how the footings would be laid at the end of a detached wall or rectangular pier.

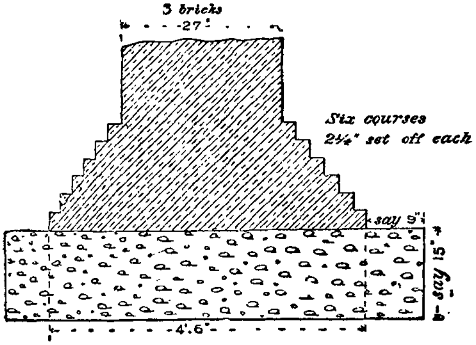


Fig. 67.—Foundations of Three-brick Wall.

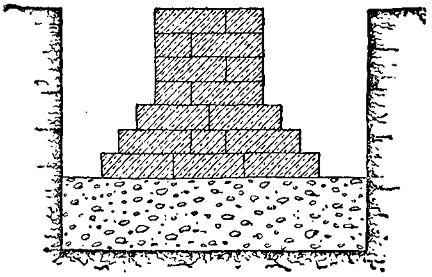


Fig. 68.—Trench for Footings of Brick-and-a-half Wall.

Footings Required by the London Building Act.

The requirements of the London Building Act, 1894, as regards footings are appended. First schedule, paragraph 9: "Unless with the consent of the council, every wall other than a wall that is carried on a bressummer shall have footings. The projection of the bottom of the footings of every wall on each side of the wall shall be at least equal to one half of the thickness of the wall at its base, unless an adjoining wall interferes, in which case the projection may be omitted where that wall adjoins; the diminution of the footing of every wall shall be formed in regular offsets, and the height from the bottom of such footing to the base of the wall shall be at least equal to two-thirds of the thickness of the wall at its base."

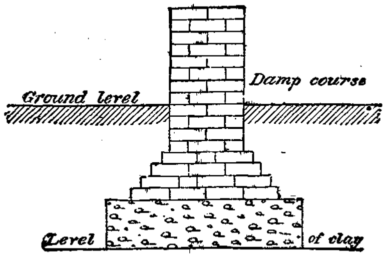


Fig. 69.—Foundation of Two-brick Wall in Light Soil.

Depth of Walls in Ground.

It is advisable to carry the masonry or brickwork of a building well below the ground, especially in a clay soil, to avoid the disturbances caused by alternate expansion and contraction upon changes of hygrometrical condition—that is, on variation of the degree of moisture contained in the soil; also, in light soils, to minimise the squeezing out of the material from under the foundation by reason of the load on it. The minimum depth depends on the soil and on the nature of the building and general circumstances; it might be, say, 2 ft. 6 in. on gravel and 5 ft. on clay; and the minimum to suit any case having been determined, it should be insisted upon unless it could be shown that there was good and sufficient reason for altering the decision.

Typical Footings.

A number of illustrations will now be given showing the footings of brick walls. Fig. 67 (scale, ⅜ in. = 1 ft.) gives a section at the base of a three-brick wall in accordance with the Building Act. Fig. 68 (scale, ½ in. = 1 ft.) shows a brick-and-a-half wall with 9-in. concrete foundations, the total depth below surface being 2 ft. 6 in. Fig. 69 (scale, ¼ in. = 1 ft.) shows an 18-in. brick wall with a concrete foundation resting on stiff clay 3 ft. below the surface. Fig. 70 (scale, 1 in. = 3 ft.) shows the base of an external wall of a dwelling-house built on a clayey soil. Fig. 71 (½ in. = 1 ft.) represents the section through the base of a wall built in double Flemish bond, and having the bottom course of footings double.

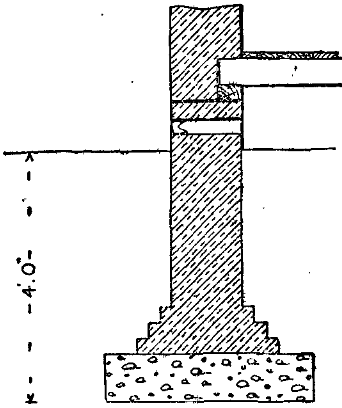


Fig. 70.—Deep Wall Built on Clay.

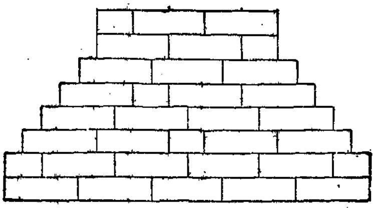


Fig. 71.—Wall Footings with Bottom Course Double.

Cavity Wall.

An example of the foundation and footings of a hollow or cavity wall are shown in Fig. 72. It is absolutely essential to have air-bricks or ventilating grids in the outer face of a cavity wall. They should be placed about 6 ft. or 8 ft. apart in the lowest course of the face-work wall, that is, at the bottom of the cavity, and there must be a damp-proof layer one course higher on the inner wall, as shown in the accompanying section (Fig. 72). Similar air-bricks must be placed at the top of the cavity to allow of continuous ventilation.

Foundation for Public Hall built on Wet Sand.

Assume the case of a public hall of two floors, to be built upon a foundation consisting of 7 ft. of dry firm sand and 3 ft. of very wet sand overlying stiff blue clay exceeding 8 ft. thick. Building on sand is proverbially risky, and in this instance would not be safe, as any drainage of the wet sand, which might take place at any time from distant operations, would probably cause a settlement and fractures in the building. The foundations should be constructed as described below. Figs. 73 and 74 show respectively a transverse and a longitudinal section of cement concrete piers with what is virtually an arch turned between each pair. The corner piers may be larger than indicated, say 6 ft. square, to act as stop abutments to the thrust. Figs. 75 and 76 show similar sections for a pile and concrete foundation. Both these methods have been used successfully; the latter method would be the cheaper, but might not be so permanent.

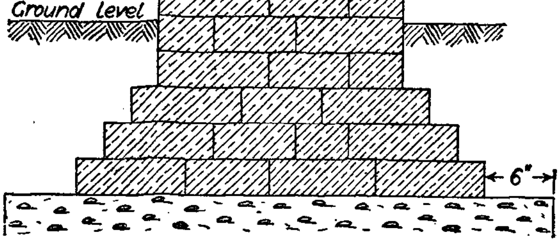
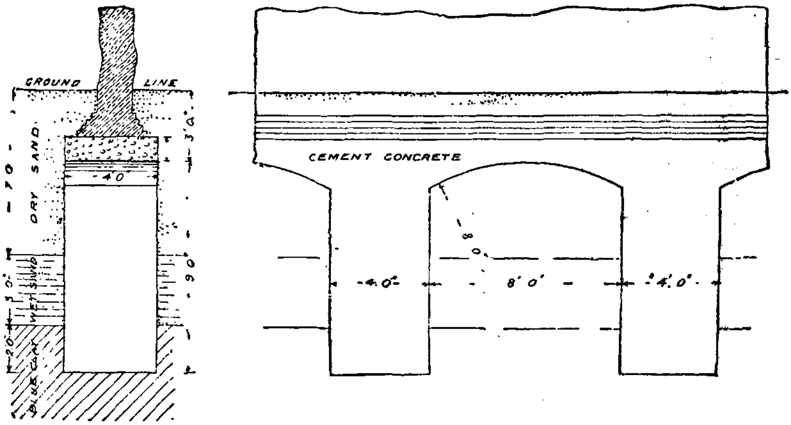


Fig. 72.—Foundation of Cavity Wall.

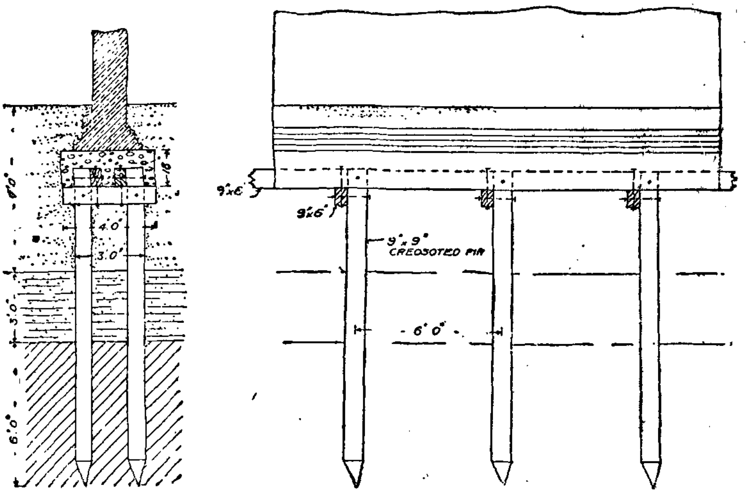
Foundations for House with Bay Windows.

Footings are usually carried round a bay window, and not straight across the opening.

but circumstances may render either method preferable. In such a case an invert arch should be formed below the opening. In building on clay the bay should be tied into the main walls by hoop-iron bond, and the main walls should be tied in the same manner to the cross walls and party walls.


GROUND LINE
DRY SAND
WET SAND
BLUE CLAY
7'0"
3'0"
2'0"
3'0"
9'0"
4'0
CEMENT CONCRETE
8'0"
4'0"
8'0"
4'0"

Figs. 73 and 74.—Cement-Concrete Piers with Arches.


6'0"
3'0"
6'0"
18"
4'0"
3'0"
9"x6"
9"x6"
9"x9"
CREOSOTED PILE
6'0"

Figs. 75 and 76.—Pile and Concrete Foundation in Bad Ground.

Foundations for Retaining Wall.

Information will now be given on foundations for a retaining wall having a base 10 ft. wide when the substratum is (*a*) blue clay, (*b*) running sand, (*c*) compact sand and gravel. A thickness of 10 ft. at the base would mean a very heavy wall. Fig. 77 shows the foundation on blue clay where the foundation must be kept well below the surface. Fig. 78 shows the foundation on running sand where the sand must be enclosed by sheet piling before excavation can be attempted; this case is one requiring very careful consideration of the whole circumstances of the site. Fig. 79 shows the foundation on compact sand and gravel, which is the best material for building upon.

Monier and Hennebique Ferro-Concrete.

Within the last few years, considerable modification in the use of concrete has been introduced by the insertion of iron rods or bars on the Monier and Hennebique systems, known also as ferro-concrete, re-inforced concrete, armoured concrete, beton fretté, etc. The earliest application of the principle appears to be due to a gardener of Paris named Monier, in 1861, hence the name given to the system. Previously, in building a warehouse for heavy goods on a doubtful foundation, it was customary to cover the site with concrete of such a depth that lines of 45° from the bottom edge of the ashlar, stone, or brickwork under the columns or stanchions, would meet at the under surface of the concrete, as in Fig. 80, to avoid the possibility of punching a hole through when the full load was on. This required in some cases a depth of 5 ft. or 6 ft., and involved considerable expense. Now the depth is reduced to one-half or less by using old rails, rolled joists, or even bars or rods embedded in both directions, as in Fig. 81.

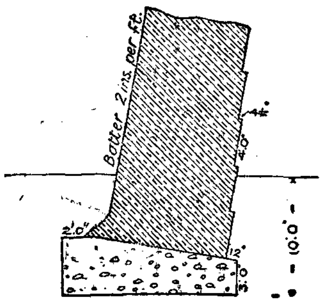


Fig. 77.—Retaining Wall Foundation on Blue Clay.

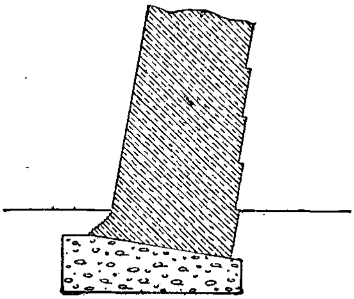


Fig. 79.—Retaining Wall Foundation on Compact Sand and Gravel.

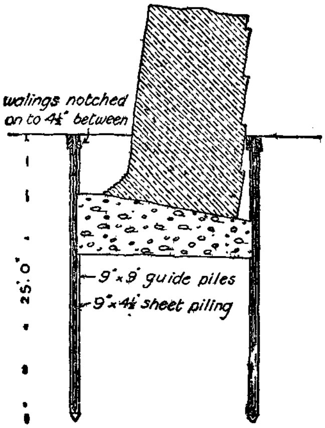


Fig. 78.—Retaining Wall Foundation in Running Sand.

The Monier System and Grillage Foundations.

The Monier system of strengthening concrete consists of rolled joists in both directions embedded in a concrete foundation on bad ground. The system is very largely used in the high buildings of America, where the joists are placed nearly or quite close together, as shown in Figs. 82 and 83. In England the system is sometimes adopted when a concrete raft is laid over the whole site of a heavy building, the joists being spaced 4 ft. to 5 ft. apart, as shown in Fig. 81. The foundations shown in Figs. 82 and 83 are known as grillage foundations. The load on the stanchions is so great that it is practically taken by the rolled joists alone until the bottom row is reached, where the spreading is very considerable, compared with

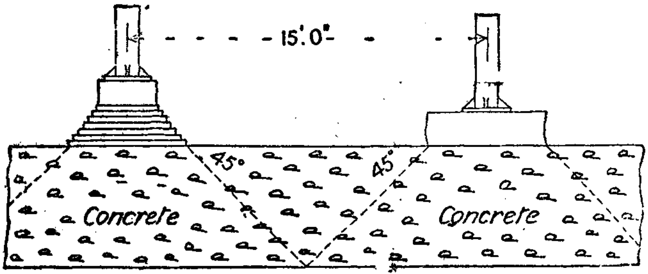


Fig. 80.—Concrete Foundation for Heavy Building.

the stanchion area, and then it is transmitted through the concrete to the subsoil. Fig. 83 shows half-elevation and half-plan of two methods of using the concrete with the same joists. The Monier system, of course, is only used when the load is so great that no other means of dealing with it would be satisfactory. The cost is considerable.

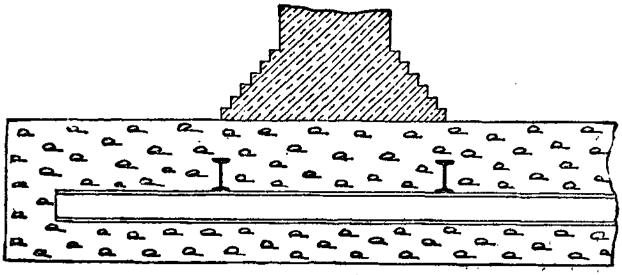


Fig. 81.—Steel Joists embedded in Concrete Foundation.

Hennebique Ferro-concrete System.

The Hennebique system is particularly applicable to piles, and consists of a combination of Portland cement concrete and steel rods. In making one of the piles, a timber mould, of an inside section corresponding to the size of the pile required, is erected and securely fixed in a suitable scaffolding. One vertical face of this mould is kept open. After the mould has been erected and everything proved perfectly plumb, the shoe or pile is placed in the bottom of the mould. In this shoe the top ends are bent inward in order to grip the concrete properly. The vertical steel rods are then placed in position, and distance pieces dropped over these rods from the top. The Portland cement concrete is then handed to the workman, who tips the concrete into the mould in quantities just sufficien[illegible] enable him properly to ram the concrete round and between the rods and distance pieces. As he gets higher with this concrete, the face of the

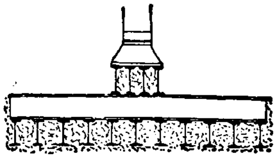


Fig. 82.

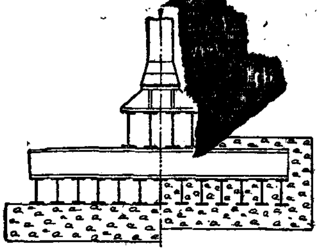


Fig. 84.

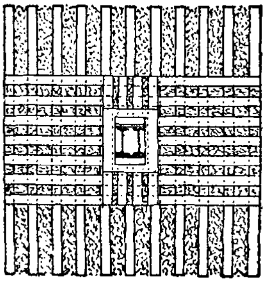


Fig. 83.

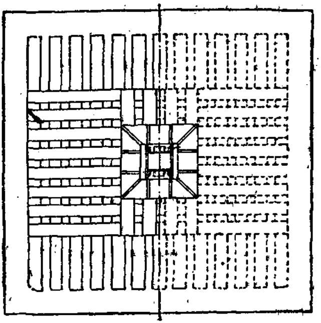


Fig. 85.

Figs. 82 to 85.—Grillage System of Foundations for Steel Stanchions.

mould, previously mentioned as having been left open, is gradually closed by suitable shutters, which are fixed by the workman as he proceeds, and the process of ramming is continued until the mould is quite filled. After about three days the moulds are stripped from round the concrete, which, though not absolutely set, is sufficiently so to retain the form given by the mould. After being moulded, a period of from twenty-eight to forty days is allowed for the pile to dry. When dry, the pile can be driven by the same methods as those employed in driving timber piles, except that extra protection is given to the head by means of a cap, in order to reduce the effect of the blows of the ram. The Hennebique system was used for the pile foundations at the London and South Western Company's dock at Southampton.

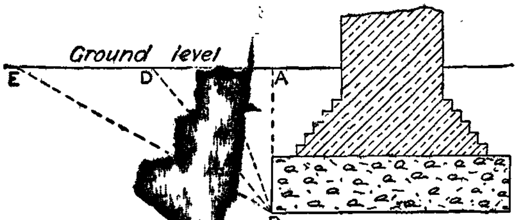


Fig. 86.—Diagram Illustrating Resistances of Various Soils.

Safe Load on Different Soils.

Tables of safe loads on soils are not of much practical value, because, after all, the question in any given case is, which of the soils named in the table corresponds with the soil that is about to be built on, and ultimately the question is one of practical judgment. Surface clay ought not to be built on at all. Deep clay, the foundations being not less than 10 ft. from the surface, also gravel and compact earth, may be loaded with as much as 3 tons to the square foot. The piers of the Charing Cross railway bridge resting on the London clay give a load even reaching 5 tons per square foot, but the depth is considerable. Light earth and sandy loam will carry from ¾ ton to 1½ tons per square foot, according to condition and circumstances. Chalk in its natural undisturbed position, simply levelled for building on, would come under Rankine's description of very soft rock, for which he gives a safe load of 1·8 tons per square foot; but there need be no hesitation in loading it up to 3 tons per square foot. If the chalk be chalk-rubble, put in, as is sometimes done, to form a foundation on loose earth, the safe load would not exceed one-fourth of the above, and might be less. The load at which settlement would be likely to take place could only be estimated after a careful examination of the site and all surrounding conditions. Good solid gravel in a layer of a thickness not less than twice the width of the base of the footings may be looked upon as nearly equal to solid rock. In practice a general maximum of 5 tons per foot super. may be taken. Where chalk underlies the gravel, without clay between, the pressure might be increased to 6 or 7 tons without danger; and where it occurs in layers with clay between, it should be reduced to 3 tons. If there is much sand in the gravel, and a possibility of water percolating through, the foundation will be treacherous, and should be spread so as to reduce the pressure to 1¼ tons per square foot. The underside of the foundation should in no case be less than 2 ft. 6 in. below the level of the surface.

Safe Load on "Made Ground."

When first depositing the material it should be well punned at every foot in depth; it will then, under ordinary circumstances, support a maximum load of, say, 1 ton per square foot. If made ground at any depth has not been punned in layers, the buildings upon it will settle considerably. Before putting in foundations a 1-cwt. rammer, worked by two men, should be used, and the base of the wall extended by concrete or otherwise to limit the weight to ½ ton per foot super. If the depth is considerable, it may be better to pile the foundation and get a bearing on the firm substratum.

Bearing Power of Various Soils.

The bearing power of various kinds of soil in pounds per square foot is given in Patton's "Treatise on Civil Engineering" as follows: Soft silt = 0, ordinary clay = 3,000, compact clay = 5,000, sand and gravel = 5,000. Another writer says, "Owing to the greater facility the outside of a foundation has for escaping pressure from a superincumbent mass, there is reason to suppose that the pressure will not be uniform; taking the average pressure as p, the maximum in centre will be about eight-fifths of p, and the minimum round the sides about four-fifths of p. Experiments on a large scale upon the supporting power of alluvial soil in India, by Mr. H. Leonard, showed that with a pressure of 1 ton per sq. ft. there was practically no settlement, but at 2 tons per sq. ft. the settlement was very decided, and sufficient to cause bad cracks if the work bonded together did not produce a uniform pressure on the foundations." Upon gravel a general average of 5 tons per ft. super. may be taken as safe, but where beds of clay are interspersed the pressure must not exceed 3 tons, and where chalk underlies the gravel the pressure may be increased to 7 tons if desired. Upon clay, in shallow foundations, no limitation of pressure will prevent movement due to expansion and contraction caused by changes of humidity; but in deep foundations a pressure of 5 tons per ft. super. may be allowed, which, as has been said, is the original load upon the columns supporting the Charing Cross railway bridge in the London clay. The Campanile of Cremona, 395 ft. high, stands on Pliocene gravel, and the pressure at its base is 12 tons per sq. ft. In a road bridge of 130-ft. span erected near Bristol, the piers consist of two wrought-iron cylinders filled with concrete and sunk through clay for about 25 ft. to a bed of hard gravel, and the pressure amounts to 5½ tons per square foot, neglecting the friction between the sides of the cylinder and the clay. By experiments at the Champs de Mars, Paris, cast-iron blocks weighted to 5·43 tons per square foot rested safely on the ground; loaded to 7·31 tons per square foot they settled; and loaded to 8·41 tons per square foot they sank out of sight.

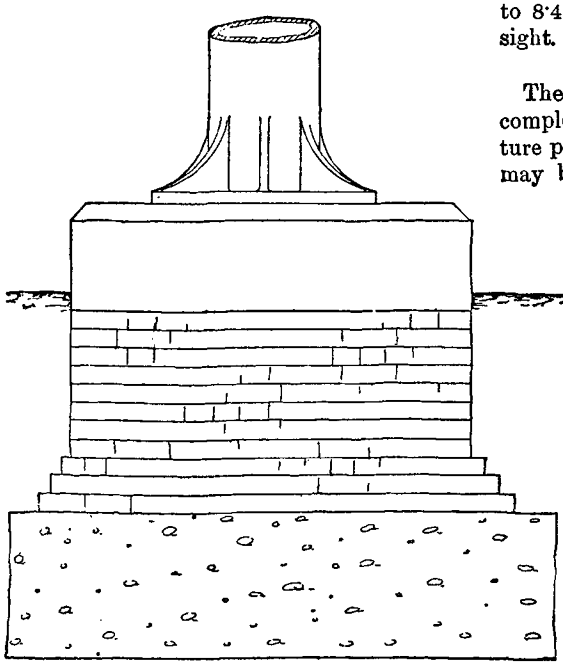


Fig. 87.—**Heavy Column Foundation in Compact Gravel.**

Rankine's Foundation Rule.

The question of stability of foundation is a complex one. The under surface of the structure producing the pressure on the foundations may be apparently acting uniformly, because the weight above may be uniformly distributed; but as the material supporting the weight can escape more readily at the sides than in the centre, there is reason to suppose that the pressure will not be uniform. Taking the average pressure as p, the maximum in the centre will be, as already said, about $1·6p$, and the minimum round the sides about $0·8p$. The average pressure is usually allowed for. Rankine's formula is based on

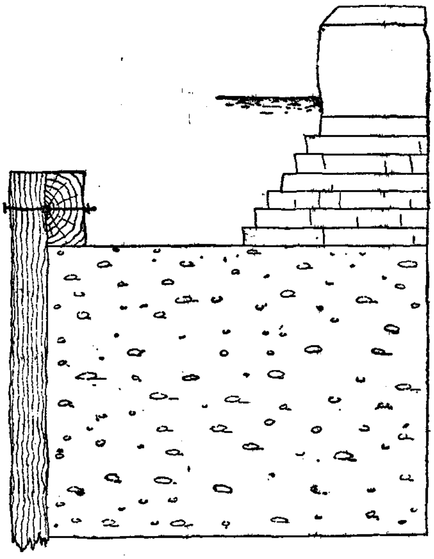


Fig. 88.—**Heavy Column Foundation in Running Sand.**

the natural slope or angle of repose of the various soils, as their supporting power will vary with their tendency to slip away from the pressure, and bulge up outside. For example, in the formula, $P = ux\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^2$, $P =$ maximum vertical pressure in pounds per square foot, $x =$ depth below surrounding surface in feet, $w =$ weight per cubic foot of earth in pounds, $\phi =$ angle of repose of earth, safe load $= \frac{1}{3}$ P. Take the case of a heavy ballast, say $w = 120$ lb., $x = 4$ ft., $\phi = 45°$, then $P =$ 7·27 tons, and the safe load = say, 2½ tons per square foot. The great difference produced by the variation in the angle of repose, or, in other words, the firmness of the soil, is shown by the change in value of the expression $\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^2$, which for ϕ 15° = 2·89, 30° = 9·00, 45° = 33·94, 60° = 193·8. An approximate formula by the writer is as follows :—$P = \frac{\phi^2}{4000} \times d$, where $P =$ the safe load in tons per square foot on the base of the foundation, $d =$ depth in feet below the surface immediately surrounding, $\phi =$ angle of repose or natural slope of the earth in degrees. The formulæ are applicable within all ordinary limits. It may be remarked that the natural slope measured from the vertical at the base of the foundation may give some measure of the resistance that has to be overcome before the foundation will sink. Thus in clay having a natural slope of 20°, only the wedge A B C (Fig. 86) would have to be overcome, while in common gravel with a natural slope of 40°, the wedge A B D would resist, and in hard compact gravel the wedge would be as large as A B E. The case is really by no means so simple as this, but the suggestion may perhaps give a little idea of the conditions.

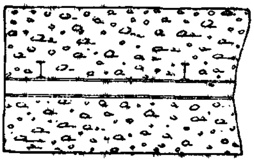


Fig. 89.—Steel Joists in Concrete Foundation.

Foundation for Heavy Column.

Figs. 87 and 88 show the foundation for the foot of a cast-iron column 18 in. in diameter, carrying a load of 120 tons, when the substratum is (*a*) compact gravel and sand, and (*b*) running sand respectively. For the first case (*a*) it is assumed that the safe load on hard sandstone or Portland stone is 15 tons per foot super. ; 120 tons load, then $\frac{120}{15} = 8$ sq. ft. area of bearing surface, or flange on base of column, say 2 ft. 10 in. square. If ordinary sandstone be employed with a safe load of 12 tons per foot super., the area will be $\frac{120}{12} = 10$ sq. ft., and the size of the flange say 3 ft. 2 in. square. The safe load on stock brickwork in mortar is 4 tons per foot super. ; therefore $\frac{120}{4} = 30$ sq. ft. area will be required for stone resting on brickwork say 5 ft. 6 in. square chamfered on upper edges. Then the projection of the stone beyond the base of the column being 1 ft. 4 in. to 1 ft. 2 in., the thickness should not be less than 1 ft. 6 in. Compact gravel and sand may be loaded with 2 tons to 3 tons per square foot, $\frac{120}{2} = 60$ sq. ft. ; therefore, the concrete should be 7 ft. 9 in. square, and should not be less than 2 ft. thick.

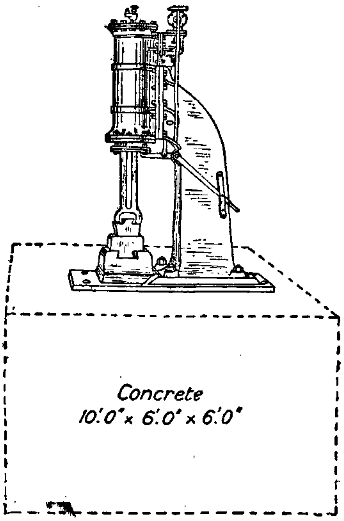
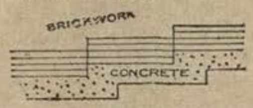


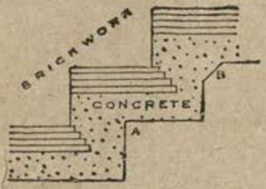
Fig. 90.—Foundation for Steam Hammer.

This will make the details as shown in Fig. 87. For the second case (*b*) the concrete should be extended to reduce the pressure to ¾ ton per square foot, say $\frac{120}{0·75} = 160$ sq. ft., making say

. square, with a thickness of 4 ft., and be unded by 12-in. by 6-in. sheet piling n down as far as it will go, or say 20 ft. 88). Other methods would be available he second case if a firm substratum existed. thickness of concrete might be reduced to if 3-in. by 1½-in. by 4-lb. rolled steel joists id in the centre, crossing both ways, 2 ft. , as shown in Fig. 89.



1.—Foundation for Wall on Sloping Ground.



-Foundation for Wall on very Steep Slope.

ions for 20-Cwt. Steam Hammer.

st form of foundation for a 20-cwt. andard steam hammer will depend on the nature of the soil, but possibly cement concrete 10 ft. by 6 ft. by 6 ft., in Fig. 90, would be suitable, though circumstances a framed bed of oak would be necessary. Generally speak- more solid and substantial the bed, the more nomical the working.

Foundations for House on Hillside.

If the nature of the soil is unknown, a pit may be dug or a boring made at one or more points on the site. If any springs exist their source should be ascertained and their course diverted, and catch-water drains should be formed on the upper side of the site to divert the surface-water. The trenches for the walls running up and down the incline must be stepped so that the building is at all points commenced from a level surface, the concrete and footings being arranged as in Fig. 91, or if the slope be steeper they should be arranged as in Fig. 92, the concrete being filled in with square junctions as at A or splayed junctions as at . This method will avoid any unnecessary excavation, and at the same time prevent any tendency to slip. If, however, the soil consists of beds of clay, there will be a possibility of the whole structure slipping bodily with the foundations along a line parallel with the slope and below the benching as at C D (Fig. 93), and in such a case the front wall should have piles driven along its base, particularly where any cross walls meet it, to prevent the slipping, as in Fig. 94, or the concrete may be carried deeper to form a cleat or toe in front for the same purpose. If earth is filled in behind the front wall to bring the surface to a level, this must be designed as a retaining wall; and if a clear level space is required at the back of the house, a breast wall must be put to keep back the earth on the upper slope. Any local defects discovered in the soil of the foundations must be made good by filling and ramming. Punning, or ramming over the bottoms of the trenches, is often advantageous in securing a more solid bed. If the superincumbent structure is of approximately uniform height and weight over the site, the settlement will be uniform; but if the weight is in excess on the lower side (generally to keep a level throughout the upper rooms), then there will be a tendency to unequal settlement, and the brickwork up to ground-floor level should be in cement or hydraulic mortar with close joints.

Foundations for School on Hillside.

Take the case of a block of school buildings, about 60 ft. by 50 ft., with stone walls, about 16 ft. high above the floor, which is to be erected on the lower part of a hill rising also behind the building at a slope of 1 ft. vertical to 6 ft. horizontal. The soil is a slippery clay for a depth of 6 ft., and then hard clay. Fig. 95 gives a section drawn to a scale of $\frac{1}{8}$,

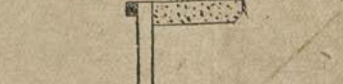


Fig. 93.—Possible Line of Rupture beneath Foundations in Clay.

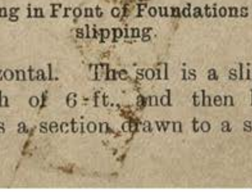


Fig. 94.—Piling in Front of Foundations to prevent slipping.

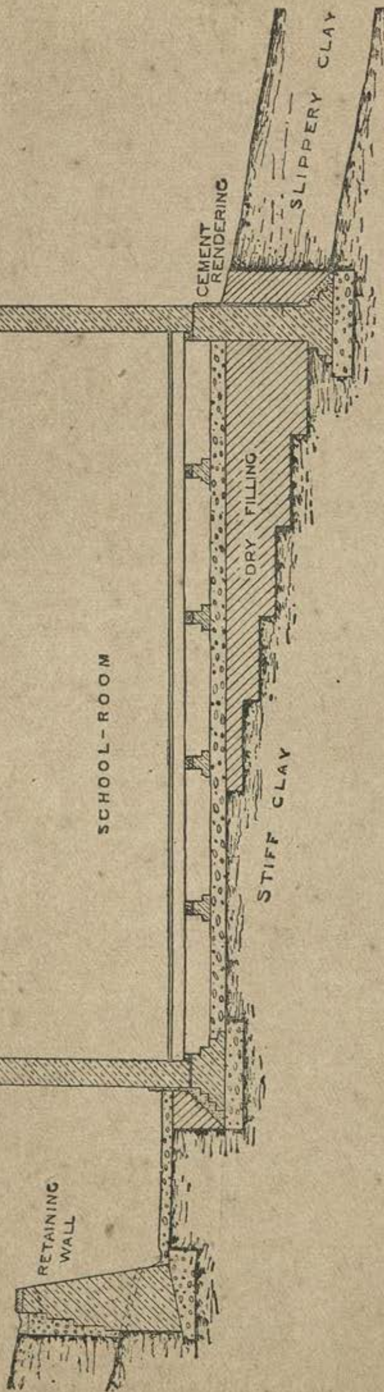


Fig. 95.—Slippery Foundation of Building on Clay.

showing the bases of the walls, the floo
provisions against damp, flooding, etc.

Foundations in Damp Soil.

Fig. 96 represents a vertical section th
the base of an outer wall of a brick dwe
house built on a damp site. The illust
shows a suitable arrangement and bondin
this is not the only one possible. Figs. 9
98 show a method of preventing the damp
in the case of the walls of a building
half basement and without a basement r
tively.

Remedying Flooded Cellar.

Take the case in which the cellar of a
house is found to be flooded, after a ver
season, to a depth of about 11 in., the
coming up through the brick floor. Assu
the existing conditions as shown in Fig. 99
up a few bricks at a convenient spot
window or grating, and sink a sump
receive the foot of a short barge pump
out all the water, and leave the pr
time in case further water collects.
soil from the outside of the wall to a
width for men to work, and down to
crete under footings, and lower if ne
keep the trench clear of water.
brickwork exposed as long as possib
out the moisture. Rake out the join
brickwork, and render in Roman cem
thick to a height of 9 in. above grou
finishing top edge level and slightly w
Portland cement (1 cement to 1 sand) may be
used if preferred. Fill in and ram the earth.
Take up the brick floor in the cellar, and
remove the earth to a depth of at least 6 in.
to the finished level and down to exi
concrete round the walls. Fill in with
land cement concrete not less than 6 in.
(1 cement, 1 sand, 1 small gravel, 4 shi
and finish with floated face in Portland ce
mortar, ½ in. thick (1 cement, 1 sand), le
the work as shown in Fig. 100. The ins
the wall may be rendered after the bric
has dried sufficiently. If it had been k
beforehand that the subsoil water would
this height, the ground should have been
drained by 2-in. agricultural pipes laid
the level of the foundations, and c
through to a well where the water cou
pumped, or to a natural watercourse wh
could flow away. If the site is in a town

perly sewered, these pipes may discharge into one of the manholes on the house side of the interceptor; the remainder of the work could then be carried out as described below.

Preventing Dampness in Basement.

Assume 4 ft. to be the level of the basement floor below outside soil. The most natural precaution would be to go to the root of the matter and drain the subsoil, if practicable, and then build in the ordinary way; but there will be a danger of causing settlements in the walls unless the drainage is very thoroughly and efficiently carried out. If in the country, a deep trench, say 6 ft. deep, may be dug all round the house at a distance of say 15 ft. or 20 ft. from it, and continued on the lower side to a watercourse. If circumstances prevent the trench being made, 2-in. agricultural pipes, without sockets and unglazed, as used for ordinary land drainage, may be laid down in trenches 6 ft. deep and 6 ft. apart across the [illegible] the pipes not quite touching (say ½-in. distance at the ends), and covered with 6 in. coarse gravel, clinkers, or stone chippings, [illegible] nothing better can be had, faggots of wood may be laid over the pipes to allow water to percolate and prevent choking. Pipes should be continued to a watercourse, flowing at a convenient level, so that ground does not remain water-logged. If the subsoil water cannot be removed, it must be excluded from the building by structural precautions, of which the first is to cover the whole site to 6 in. beyond lowest brick footing with not less than 6 in. thickness of good Portland cement concrete, rammed as

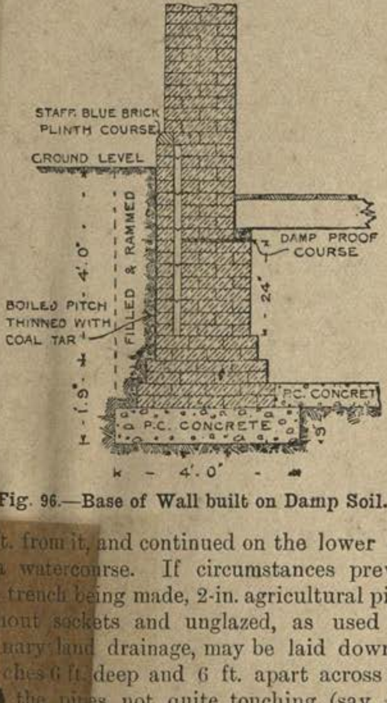

STAFF. BLUE BRICK
PLINTH COURSE
GROUND LEVEL
FILLED & RAMMED
4'.0"
DAMP PROOF COURSE
2'4"
BOILED PITCH THINNED WITH COAL TAR
1'.9"
P.C. CONCRETE
P.C. CONCRETE
9"
4'.0"

Fig. 96.—Base of Wall built on Damp Soil.

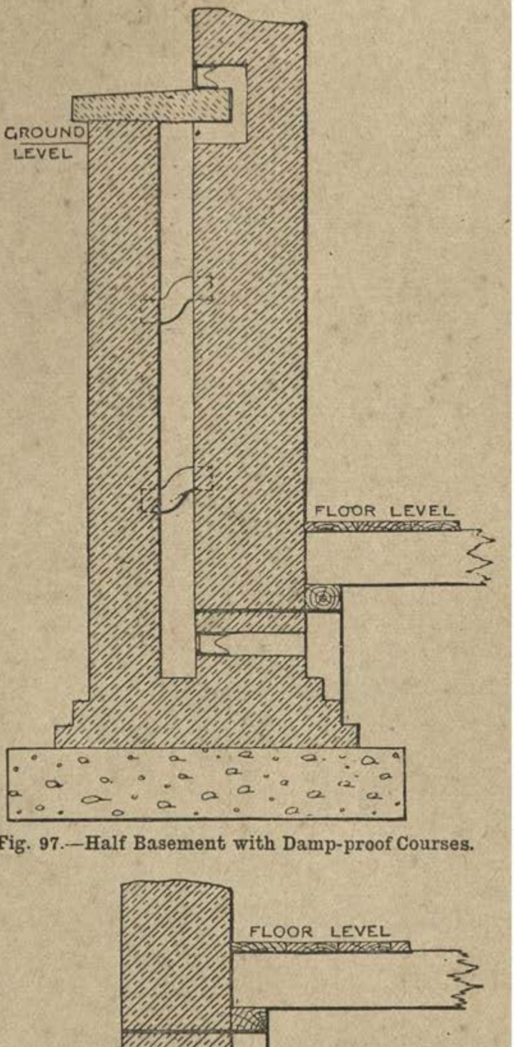

GROUND LEVEL
FLOOR LEVEL

Fig. 97.—Half Basement with Damp-proof Courses.

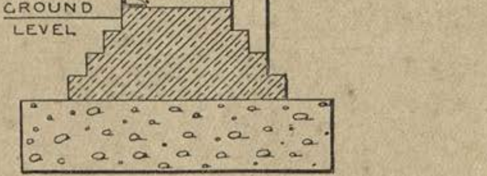
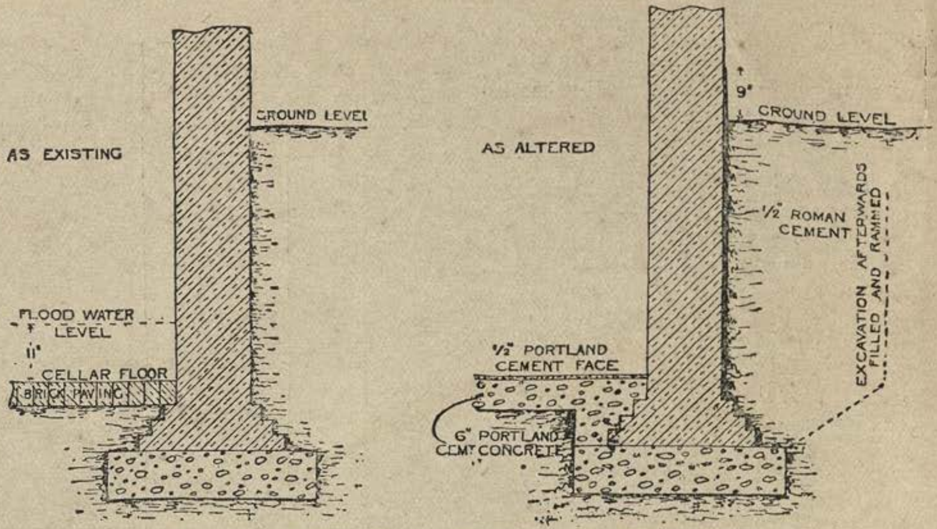

FLOOR LEVEL
GROUND LEVEL

Fig. 98.—Base of Wall with Damp-proof Course.


AS EXISTING
GROUND LEVEL
FLOOD WATER LEVEL
11"
CELLAR FLOOR
BRICK PAVING
AS ALTERED
9"
GROUND LEVEL
½" ROMAN CEMENT
EXCAVATION AFTERWARDS FILLED AND RAMMED
½" PORTLAND CEMENT FACE
6" PORTLAND CEMT CONCRETE

Figs. 99 and 100.—Altering Cellar to make it Damp Proof.

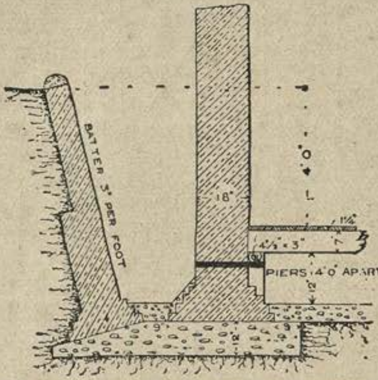

BATTER 3" PER FOOT
18"
7'·0"
1¼"
4½" × 3"
7"
PIERS 4'0" APART
12"
9"
9"
12"

Fig. 101.—Open Dry Area to Basement.

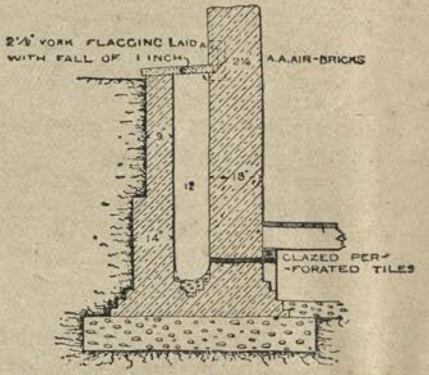

2½" YORK FLAGGING LAID WITH FALL OF 1 INCH
2¼
A.A. AIR-BRICKS
9"
12
18"
14"
GLAZED PERFORATED TILES

Fig. 102.—Closed Dry Area to Basement.

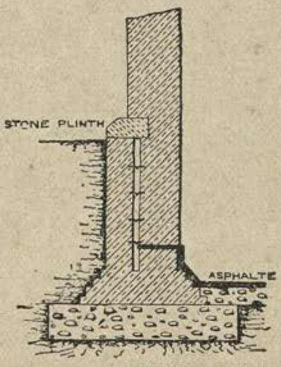

STONE PLINTH
ASPHALTE

Fig. 103.—Basement with Closed Dry Area and Stone Plinth to cover Cavity.

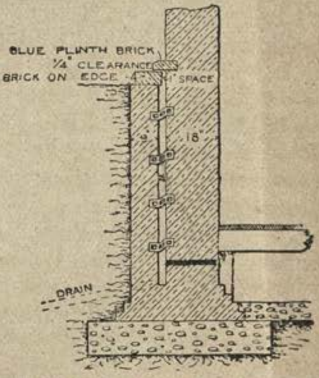

BLUE PLINTH BRICK
¼" CLEARANCE
BRICK ON EDGE
4"
1" SPACE
9"
18"
DRAIN

Fig. 104.—Dry Area with Blue Plinth Brick to cover Cavity.

laid, and, if a wood floor is not required, floated with neat cement. The trenches for foundations of walls are usually put in first, then the concrete under the footings and the surface concrete after the brick footings. The concrete under footings are made from 6 in. to 12 in. thick. Care must be taken that the earth is nowhere in contact with the brickwork of the footings. The outside earth above the level of the concrete should be kept away from the main wall by a retaining or breast wall, leaving a wide and open dry area, and in addition a cement plinth should be carried up the main wall as high as the damp-proof course. (See Fig. 101.) When the necessary space cannot be given, or other objections arise to an open dry area, a closed dry area may be substituted. This may be 12 in. wide, and the retaining wall vertical, with one set off at back. The space must be ventilated without allowing access for vermin, and the bottom must be drained by gully-traps at sufficient intervals. (See Fig. 102.)

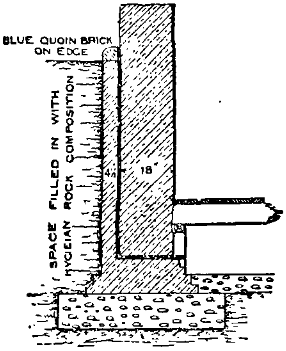


Fig. 105.—Hygeian Rock Composition in Hollow Wall.

Alternative Arrangements.

A cheaper but an inferior arrangement may be made with glazed bonding bricks to resist the thrust. Inserted as shown in Fig. 104 they prevent any moisture from passing to the main wall. Instead of the bonding bricks, galvanised iron ties may be used as in Fig. 103, although they are not so suitable for resisting thrust. The 4½-in. wall may be part of the main wall, and not extra material, and a stone coping may be used with a blue brick plinth to cover the cavity. (See Fig. 104.) Quite a recent method is the use of hygeian rock composition grouted in between the main wall and a thin outer skin, as in Fig. 105. A somewhat similar arrangement is an external vertical damp-proof course of good asphalt, as Claridge's Seyssel asphalt. (See Fig. 106.) Portland cement rendering is sometimes used in this way; it is cheaper, but less efficient. In either case the floor may be direct on the concrete if desired, and to avoid the coldness of cement face or asphalt, wood-block flooring may be used. The hardest bricks obtainable should be used wherever exposed to damp. Staffordshire blue bricks are best, owing to their thorough vitrification.

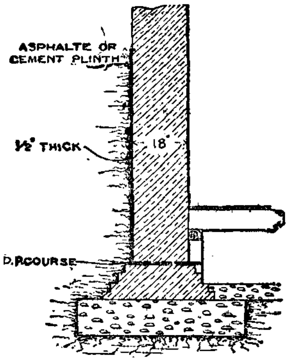


Fig. 106.—Vertical Damp-proof Course.

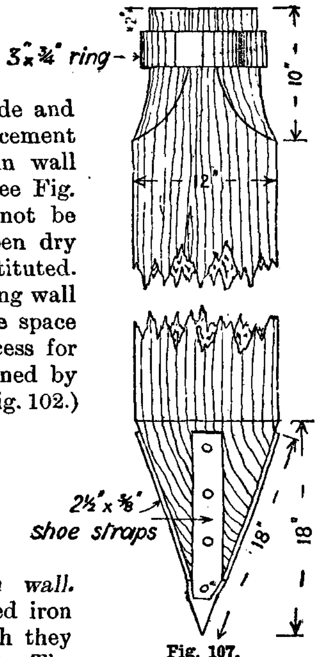


Fig. 107.

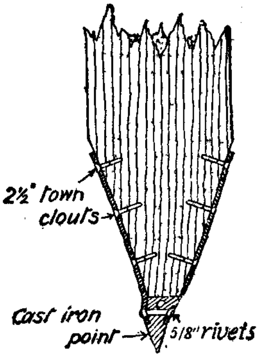


Fig. 108.

Figs. 107 and 108.—Head and Shoe of Pile.

Timber Piles.

Figs. 107 and 108 on page 53 show (one-sixteenth full size) the details of a 12 in. by 12 in. pile, with ring at head and shoe at point ready for driving. Details of another and similar shoe are given in Figs. 109 and 110. Two items of great importance are, that the points of the piles should be in the true line of the axis and that the sides should be equally bevelled. The length of each pile should be measured before it is pitched for driving, and the centering of the point should be tested from each face. The rings or hoops should not be more than $1\frac{1}{2}$ in., nor less than $\frac{3}{4}$ in., from the top, and the top 12 in. is generally specified to be cut off after driving, besides any cutting that is required to form a scarf, or any cutting to break joint. This is done in order to ensure that any bruised timber is not left in. With care the cutting off of 6 in. will often be sufficient to remove any damaged material. Greenheart piles are the best that can be used for resisting marine worms and general decay. No period can be named for the life of such piles, as the writer does not know of any that have yet failed. Greenheart piles driven into sand will not require shoes; the length of the piles can be determined only by trial.

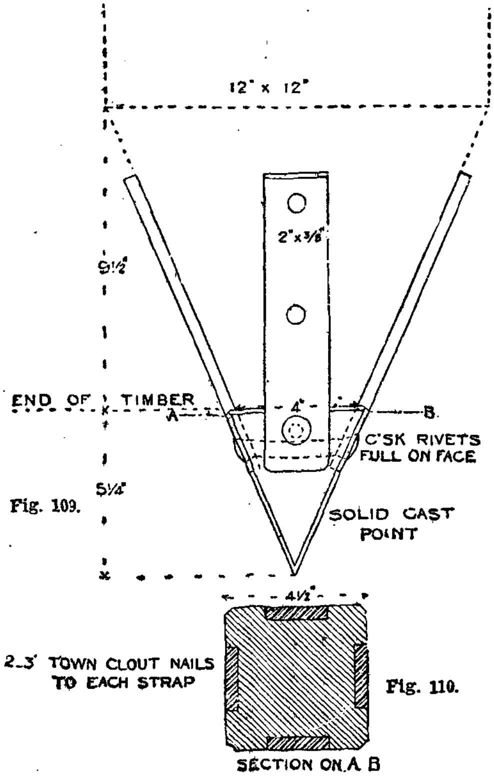


Fig. 109.

Fig. 110.

Figs. 109 and 110.—Shoe of Pile.

Pile-driving Engine.

The general arrangement usual in a small pile-driving engine worked by hand power is shown by Fig. 111. Larger ones are on the same principle, and the capacity of the boiler and winch will depend upon the price paid, but a vertical boiler with small winch engine attached will probably be suitable. Oblique piles are driven by canting the pile-engine, but the blow, of course, loses in efficiency according to the amount of cant. For moving the pile-engine about a job on shore, it is usual to lay down a pair of rails and to prise the engine along them. For transportation by water, a barge is the best means, but if by road, a lorry, or low trolley, is usual, the engine being carried erect, if there are no bridges to pass under, and being made fast by guy ropes from the top to the angles of the lorry.

Pile-driving.

Generally, it may be assumed, piles have to be driven to a given depth, but they should also be driven until the last blow does not send the piles down more than $\frac{1}{4}$ in., or such other limit as may be fixed upon. The distance that a pile is driven by a blow may be found by ruling a line across from the head of the pile with a level straightedge, and chalking the side frame of the pile-engine before and after the blow. Piles cannot break off in their length, if any care at all has been exercised in selecting the timber. When a large stone that cannot be forced on one side is met with, and the pile has been driven to half its depth, the usual plan is to leave the pile as it is; but at a less depth the stone should be removed by excavation, or a new pile should be driven alongside the first pile, and the first one withdrawn, or cut off, if the object of the piling will permit of such an arrangement. When the pile is going out of line, an experienced pile-driver can generally get over the difficulty; but if the deviation is due to the point not being axial, the only remedy is to withdraw the pile and shoe it afresh.

Power Required for Pile-driving.

A mistake is commonly made in the terms used in pile-driving. The "monkey" is the clip-hook that runs up and down the pile-engine (and hence the name) to catch hold of the ram, lift it, and then drop it. (See Fig. 112.) Many persons wrongly call the ram by this name; the ram is the hammer or tup. The pressure on the head of the pile produced by the impact of the ram depends on the distance the head of the pile moves after the ram has struck it. This takes into account the compression of the timber as well as the movement of the point of the pile. The pressure is comparatively small at the beginning of the impact, and reaches its maximum when the downward motion of the ram ceases. It is usually stated as an average—namely, energy of impact ($W h$) = pressure produced (p) × distance driven (d) —but this would be mean pressure only; the maximum would be at least double, and more probably three times, the mean. The curve of pressure would probably approximate to a semi-parabola, as in Fig. 113, which would make the maximum $p = 3\ W\ h/d$; but it is really more complex, as there will be an intermediate maximum just before the pile begins to move. If the pile "refuses," the whole of the energy will be absorbed in compressing the timber, and there are no data available for estimating the pressure produced, although the principle would be as just stated. For finding the weight of the ram required, the simplest rule is to make the weight of the ram equal to the weight of the pile. By another rule, the diameter of the pile minus 5 in. is multiplied by 3 in order to give the weight of the ram in hundredweights. Rams generally run from 10 cwt. to 20 cwt. for ordinary work. Another rule is that the weight of the ram in pounds should not be less than one-fourth of the product of the length of the pile in feet into the sectional area in square inches.

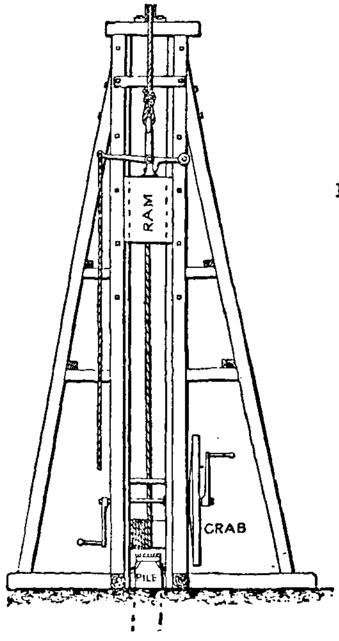


Fig. 111.—Pile-driving Engine.

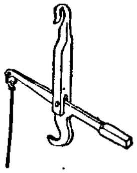


Fig. 112. — Monkey of Pile-driving Engine.

Force Exerted by Ram on Pile.

Assume that the ram weighs 18 cwt. and drops 6 ft. The effect can be estimated as follows: The foot-pounds of work in a falling weight = weight in lb. × height of fall in feet.

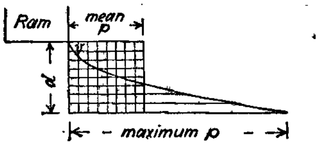


Fig. 113.—Diagram showing Pressure of Ram on Pile.

In the case quoted, 18 × 112 × 6 = 12,096 ft.-lb. The effect of this on the pile varies according to the conditions of the case, and no precise statement can be made as to the equivalent dead load. Suppose the pile to move 1 in., it is then generally stated that the average pressure produced is $\frac{12{,}096}{\frac{1}{12}} = 145{,}152$ lb., or $\frac{145{,}152}{2{,}240} = 64{\cdot}8$ tons, and that the supporting power is the same. It may be said that while a ram of 8 cwt. falling 7 ft. produces an energy of 56 ft.-cwt., and a ram of 4 cwt. falling 14 ft. produces the same theoretical energy, the practical effects are very different, the lighter ram tending to smash the head of the pile rather than drive the toe downwards. When the Lacour pile-engine is used, its blows are so rapid that the pile has not time to come to rest after each blow as in the case of an ordinary pile-engine, and the pile being kept in a constant state of vibration is driven more easily. The 6-cwt. ram of a Lacour engine falling

4 ft. 6 in. every second strikes a blow equal to 3,024 ft.-lb., and gives 181,440 ft.-lb. per minute. The 15-cwt. ram of an ordinary engine falling 8 ft., say every 10 seconds, strikes a blow equal to 13,440 ft.-lb., and gives 80,640 ft.-lb. per minute. So that the former would give out more than double the work; but with large piles there would be a probability of the blow not being heavy enough.

Safe Load on Pile.

The safe load that may be supported on a pile can be ascertained by the ordinary formula (Major Saunders, United States Engineers, quoted by Rankine and Molesworth)—

d = distance driven by last blow in inches (assume $\frac{1}{2}$ in.).

h = height fallen through by ram in inches (assume 72 in.).

w = weight of ram in cwts. (assume 15 cwt.).

Safe load in cwts. $= \frac{w\,h}{8\,d} = \frac{15 \times 72}{8 \times 0{\cdot}5} = 270$ cwt.

$= 13\frac{1}{2}$ tons.

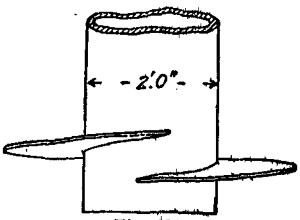


Fig. 114.

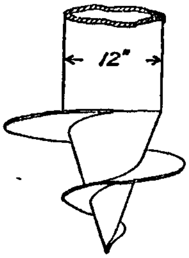


Fig. 115.

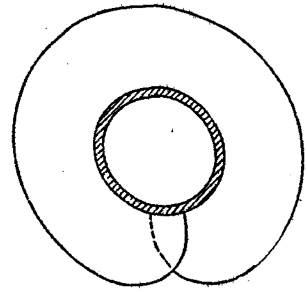


Fig. 116.

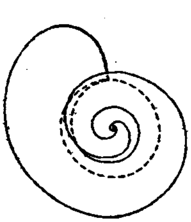


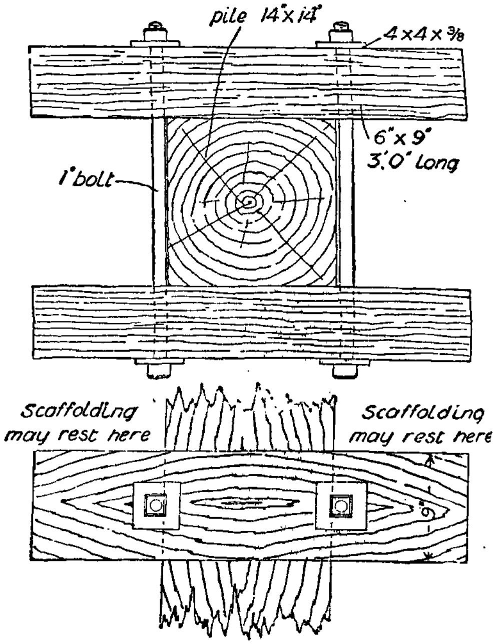
Fig. 117.

Figs. 114 and 116.—Parallel Cast-iron Pile Screw.

Figs. 115 and 117.—Tapering Cast-iron Pile Screw.

Screw Piling.

Screw piling has been much used for the foundations of bridges. The screw is usually formed in cast-iron, either parallel as in Fig. 114 and Fig. 116, or tapered as in Fig. 115 and Fig. 117. The former has a cutting edge at the bottom, and is cast in a short length to bolt to the lower end of a cast-iron cylinder. The latter is usually fixed by a cottar to the lower end of a wrought-iron or mild steel column. They are screwed in the ground by means of a wooden frame bolted to the upper end of the column rotated with capstan bars by hand power, or by the assistance of a crab winch.



Figs. 118 and 119.—Securing Scaffolding to Piles.

Securing Scaffolding to Piles.

When constructing a timber jetty it should be borne in mind that sea-worms attacking piles must enter below high-water mark, so that nail-holes above that level offer no attraction to them, but all timber exposed to the weather should have as few nail-holes as possible. Scaffolding may be supported by pieces bolted on each side of the pile, as shown in Figs. 118 and 119, without making any holes through the pile. Greenheart is generally credited with entire freedom from the attack of sea-worms, and it is only its expense that prevents its more common use.

Bridge Foundations in Running Sand.

Foundations in running sand are most difficult to construct, and instructions as to the best

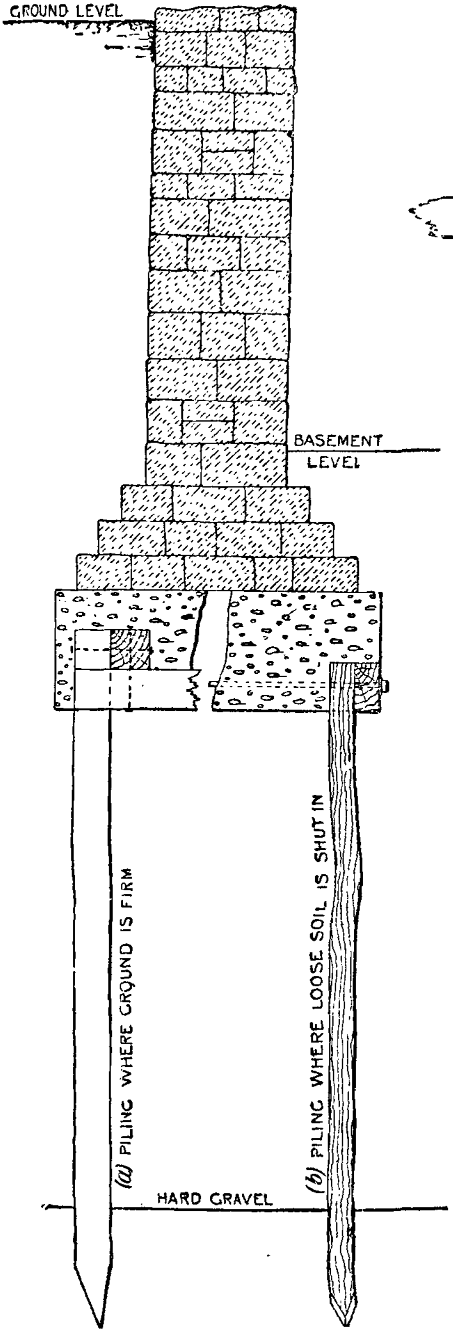

GROUND LEVEL
BASEMENT LEVEL
(a) PILING WHERE GROUND IS FIRM
(b) PILING WHERE LOOSE SOIL IS SHUT IN
HARD GRAVEL

Fig 120.—Pile Foundation in Running Sand to carry Lofty Wall.

method of setting about the work are not easily given when the nature, size, and weight of the bridge and the general circumstances are not known. If the bridge is to be of brick or stone, piles should be driven over the site, the heads being connected together by longitudinal and transverse timbers embedded in concrete. To get down to the bottom level for this purpose, sheet piling may have to be driven round the site when the running sand is reached, the dry sand being kept up by ordinary timbering, or by making the excavations with sloping sides. If no piling is done under the bridge foundations, sheet piling will be absolutely necessary, and the sheet piling should then be left in, and should be paid for as part of the foundation.

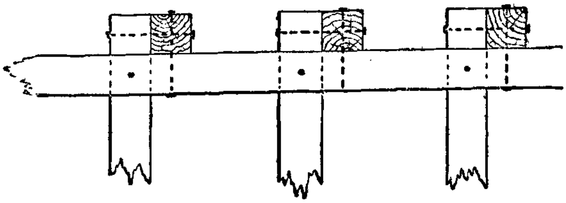


Fig. 121.—Plan of Ordinary Piling.

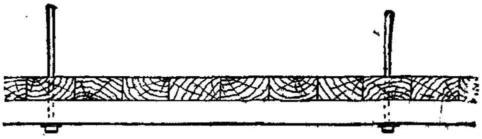


Fig. 122.—Plan of Sheet Piling.

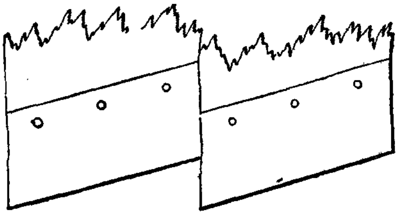


Fig. 123.—Foot of Sheet Piling.

Warehouse Foundations in Running Sand.

Assume that in getting out the foundations of a lofty warehouse, running sand is met with necessitating the driving of piles into a hard bed of gravel 16 ft. below the basement floor level, which is 9 ft. below ground level. Taking the external walls as built of coursed rubble, 3 ft. thick at the basement level, the necessary working drawings showing how to carry them where the running sand occurs, with enlarged sketches of details, will be as in Figs. 120 to 123; Fig. 120 shows the wall, its footings, the concrete, and alternative styles of piling; the ordinary piling (to the left of Fig. 120) is shown in plan by Fig. 121, and the sheet piling (to the right of Fig. 120) by Fig. 122, the foot of the sheet piling being illustrated by Fig. 123.

School Foundations on Silty Soil.

In the case of a school of two floors which has to be built on a silty foundation, the base of the walls being in piers two bricks thick, a section through one of the openings will be as in Fig. 124. The trench should be excavated to the required depth, the piles driven, the longitudes and cross timbers bolted on, and the concrete then packed in and filled over the top to the underside of the footings.

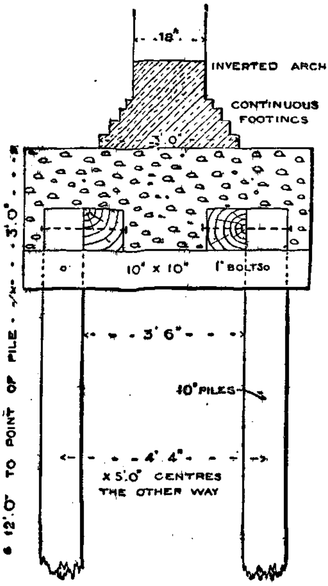


Fig. 124.—Pile Foundation on Silty Soil.

Drawing Piles.

Piles on land may be drawn by the method shown in Fig. 125. Take the case of a 14-in. pile, which has been driven some time, the final pressure being assumed by McAlpine's formula: $80\ W + {\cdot}228 \sqrt{H - 1})$, where W = weight of ram in cwt., H = fall in feet; say $W = 10$ and $H = 16$, then ultimate load $= 80(10 + {\cdot}228\sqrt{16} - 1) = 793$ cwt. or 39·648 tons. It may be assumed that a force equal to two-thirds of this amount will be necessary to extract it = 26·432 tons, or say 27 tons. In the arrangement shown, where a 24-in. beam was proposed, the beam will weigh say $24 \times 2 \times 2 \times 34 \div 2240$ = say 1½ tons acting downwards with a leverage of 12 ft., or a moment of $1{\cdot}5 \times 12 = 18$ foot-tons. The resistance of the pile will be 27 tons, with a leverage of 4 ft. or a moment of $27 \times 4 = 108$ foot-tons. The power required to be exerted by the crane at the other end of the beam will then be $\frac{18 + 108}{24} = 5{\cdot}25$ tons. If this does not start the pile it should be struck with a heavy mall or a beam used as a battering ram to set the pile in vibration. A calculation should be made to see that the beam is strong enough to act as a cantilever; thus, $\frac{\frac{3}{4}\ b\ d^2}{L} = \frac{\frac{3}{4} \times 24 \times 24^2}{20} = 518$ cwt. = 25 tons breaking weight at end, so that it has ample strength.

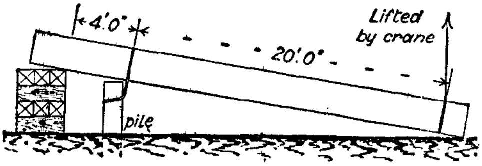


Fig. 125.—Method of drawing Pile.

An 18-in. beam would appear to be sufficient if the resistance does not exceed that given above, because $\frac{\frac{3}{4} \times 18 \times 18^2}{20} = 218{\cdot}7$ cwt., or say 11 tons at crane end to break it, but it is well to allow plenty of margin. Care should be taken that there is no large knot on the lower side of the beam near the pile, which, however, would be no detriment on the top. The chain would have to resist say 27 tons. A ⅝-in. chain would bear not more than 3⅛ tons on a single lead, and it would therefore require to be lapped round the beam at least four times, making not less than 8 leads of the chain fully strained to carry the load. All these assumptions and calculations are, of course, approximate, as precise rules cannot be laid down for such work.

BRICKWORK.

Bond Defined.

Bond in brickwork is the arrangement of the bricks so that the joints in one course are covered by bricks in the adjacent courses, and continuous vertical joints are avoided. The arrangement may be varied according to the appearance required on the two faces. It is usual to lay as many headers as possible in the interior, except that in thick walls diagonal or raking bond is occasionally used. A straight joint means imperfect bonding wherever it occurs; in exposed face-work of course it can never be allowed. A brick-on-edge coping is the only part where an occasional straight joint or the face cannot be prevented.

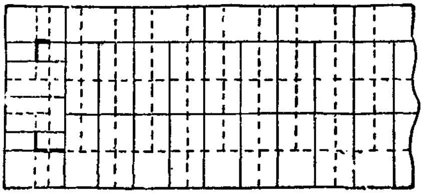


Fig. 126.—Two-and-a-half Brick Wall in English Bond.

Hoop-iron Bond.

Hoop-iron bond is either a plain band of iron, such as is used to fasten bales of goods, about 1 in. wide by No. 20 gauge thick, or it is stouter, and specially made with triangular stabs in it to cause projections, as in Tyerman's patent. In either case it is usually tarred and sanded, and then laid in the courses of brickwork parallel with the face, one to each half-brick thickness of wall, and at such intervals in height as may be directed by the architect. The object is to strengthen the wall, especially where settlements are liable to take place. Sometimes it is laid in footings only, at other times at the angles of a building; and again it may be used as a virtual stringcourse around a building between the successive floors. The only disadvantage that could be caused by its use would be due to rusting if insufficiently protected and laid in a damp wall.

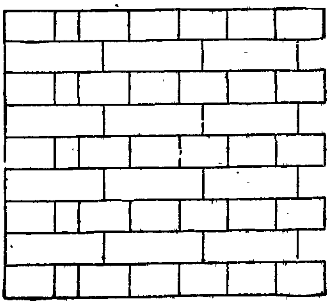


Fig. 127.—English or Old English Bond.

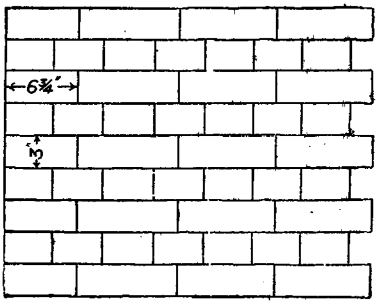


Fig. 128.—English Bond without Closers.

English Bond.

English bond consists of alternate courses of all headers and all stretchers showing on face, except at the end, where the heading course has usually a closer next but one to the end. Sometimes, instead of the closer in this course, a three-quarter brick is used to begin the stretching course. The interior of a thick wall in any bond is usually filled with headers, but sometimes with alternate courses of diagonal or

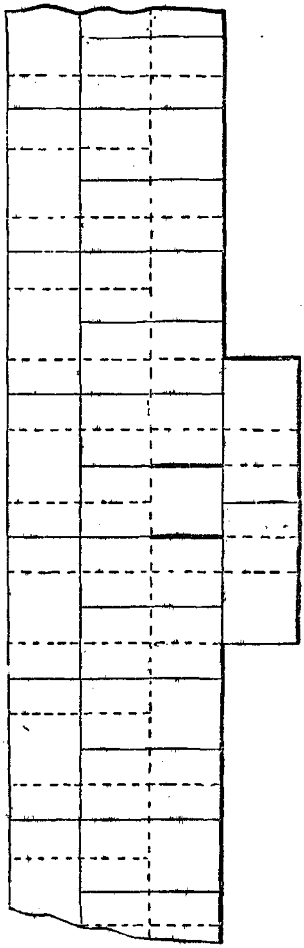


Fig. 129.

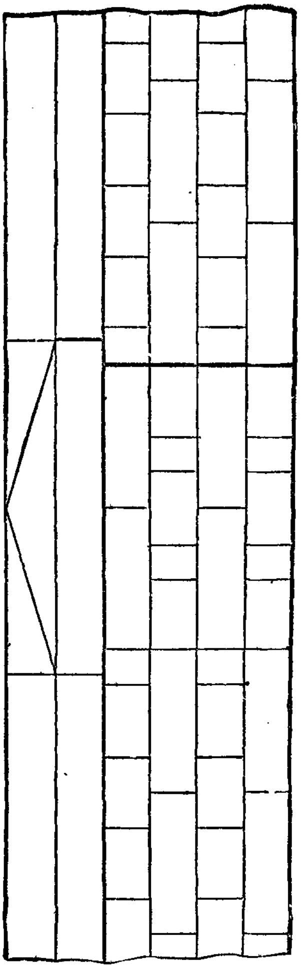


Fig. 131.

Figs. 129 to 131.—Brick Boundary Wall in English Bond.

raking bond. Fig. 126 shows the arrangement of the bricks in a 2½-brick wall built in English bond, the bricks in the course below being indicated by dotted lines. Elevations showing English bond are given in Figs. 127 and 128. Figs. 129 to 131 show a boundary wall in English bond, finished with a stone saddleback coping, which is 6 in. deep, 3 in. wider than the wall, and is weathered down 3 in. The dotted line in Fig. 130 shows how the coping is shaped to throw rain clear of the face of the wall.

Old English Bond.

There is no difference, so far as the writer is aware, between English and Old English bond;

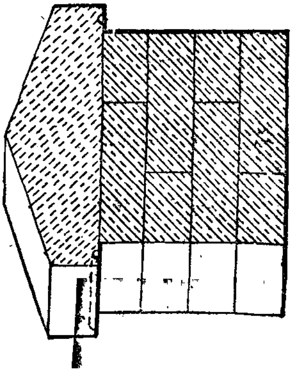


Fig. 130.

the names are used indiscriminately for the same thing. In Scotland some of the methods of bonding brickwork may differ from the ordinary method in use elsewhere; but Scotland is not a brick country, and the southern methods ought to prevail. There is no distinction between the two in London, where a wall showing on the face alternate courses of all headers and all stretchers is called indifferently by either name. In the manufacturing districts of the North of England Old English bond is held to mean a wall showing three courses of stretchers to one of headers.

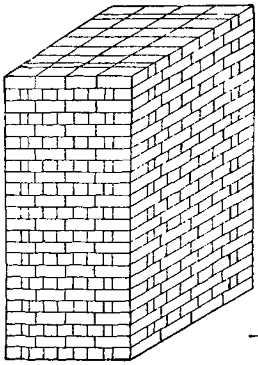
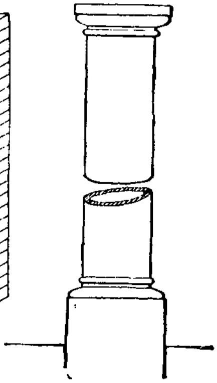



Fig. 132.—Brick Pillar. Fig. 133.—Iron Column.

Bond Timbers.

Bond timbers are liable to shrink and allow a settlement of the wall. They may also swell by access of moisture, and loosen the brickwork. As they are built in on three sides, the free access of air is prevented, and they are liable to dry rot, and thus leave some of the brickwork unsupported. The chief objection, however, is that in case of fire they are liable to be consumed and cause the walls to fall, with the risk of burying firemen and others in the ruins. They were extensively used at one time because wood was plentiful and cheap and only common bricks and mortar were available, and they formed a convenient longitudinal tie. Since the Great Fire of London in 1666 hoop iron has taken the place of bond timbers to an increasing extent.

Pillars and Columns.

A pillar may be any column, pier, stanchion, or storey post of brick, stone, iron, or wood. The brick pillar is illustrated by Fig. 132. The term column (see Fig. 133) may be used synonymously with a pillar, but is more often restricted to stone or iron of more or less cylindrical form.

Brick Pier in English Bond.

The bonding of the 3½-brick pier shown in Figs. 134 and 135 has the advantage of not containing any vertical through joints in the mass, but there is the great disadvantage of necessitating fourteen three-quarter bats in every course, none of the pieces cut off being required as closers. The bonding shown in Figs. 136 and 137 would involve much less waste of good bricks, and would have the same external appearance. The vertical through joints are shown by doubled lines in Fig. 137. All the courses will be bonded alike, but each upper one turned one-quarter round. By so doing, the vertical through joints shown by the dotted doubled lines will run only two courses. There is no standard of bonding for a 3½-brick pier, and some latitude may therefore be allowed to individual opinion. In Figs. 138 and 139 another method is shown, with no vertical

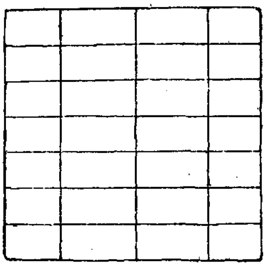


Fig. 131.

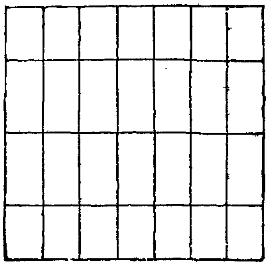


Fig. 135.

Figs. 134 and 135.—Courses of Three-and-a-half Brick Pier in English Bond.

through joints, but with more closers. Each successive course in this case would require turning one-quarter round. Figs. 140 and 141 show two courses of a four-brick pier in English bond. If in an exposed situation and subject to rough usage, the angle or quoin bricks should be blue Staffordshire bullnose bricks. Some misapprehension exists as to the height a brick pier could be built without a load on top. Assuming the safe load on a course of bricks to be 8 tons per square foot, and taking the weight of a cubic foot of brickwork at 120 lb., the safe height of such a pier, neglecting wind pressure and the additional stresses due to incipient bending, would be

$$\frac{8 \times 2240}{120} = 149\tfrac{1}{3} \text{ ft.}$$

but this is practically absurd. The strength of brick pillars begins sensibly to reduce when the height exceeds six times the least diameter or width. Nominally the safe load may be one-tenth the ultimate load, or one-third of the load at which fracture may begin, but practically much lower values are taken for the working load owing to the unusual precautions taken to reach a high figure in testing and the large contingencies that arise in practice. A practical rule to allow for varying heights of brick piers is

$$S = \frac{24\,W - Wr}{18},$$

where S = safe load tons per ft. sup. on given brick pier, W = safe load tons per foot sup. on cubical block of brickwork, r = ratio of height of pier to least thickness. This would

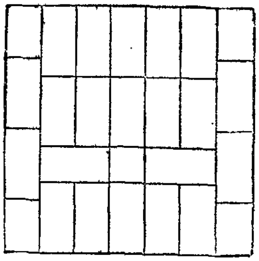


Fig. 136.

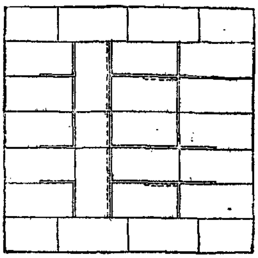


Fig. 137.

Figs. 136 and 137.—Brick Pier Bonding: Alternative Arrangement.

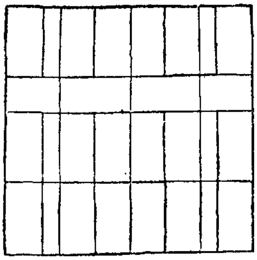


Fig. 138.

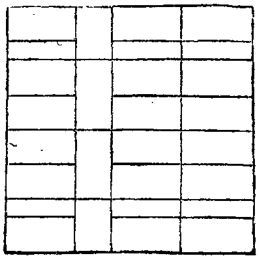


Fig. 139.

Figs. 138 and 139.—Brick Pier Bonding: Alternative Arrangement.

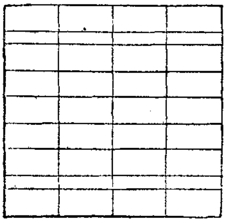


Fig. 140.

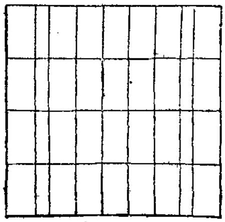


Fig. 141.

Figs. 140 and 141.—Four-Brick Pier in English Bond make the height of the four-brick pier about 53 ft., but even then it would be useless, as it would have to be kept out of the draught to enable it to carry its own weight.

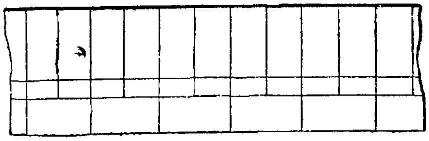


Fig. 142.

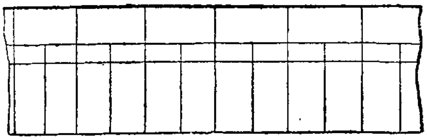


Fig. 143.

Figs. 142 and 143.—Bonding Brickwork for Plinth.

Bonding Brickwork for Plinth.

Figs. 142 and 143 show the bonding of brickwork in English bond for the formation of a plinth with 2¼-in. projection in a 1½-in. brick wall. Fig. 144 shows the same wall in section with footings and concrete.

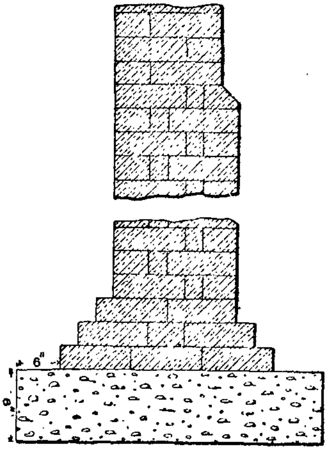


Fig. 144.—Elevation of Wall with Plinth.

Flemish Bond.

When a wall is described as being built in Flemish bond, the appearance on both faces is meant, single Flemish relating to the external face only. Flemish bond consists of alternate headers and stretchers in every course—that is, each course contains both headers and stretchers, the headers of one course being central between the stretchers of the adjacent courses. Figs. 145 and 146 show the bonding of a 1½-brick wall in single Flemish bond—that is, Flemish facing with English backing—with snap headers in alternate courses. The thick lines show the straight joints in the interior of the work, which are a source of weakness, but the snap headers save 12½ per cent. of the extra quality facing bricks. Figs. 147 and 148 show the bonding for the same wall in double Flemish bond, and the thickened parts show the straight joints as before. The two systems have the same percentage of straight joints, but the latter will be rather stronger owing to the straight joints being more uniformly divided over the whole wall. Fig. 149 shows an elevation of a wall in Flemish bond.

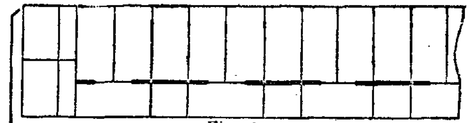


Fig. 145.

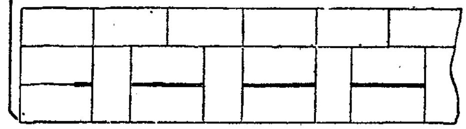


Fig. 146.

Figs. 145 and 146.—Courses for Single Flemish Bond.

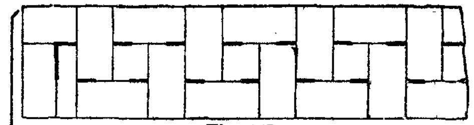


Fig. 147.

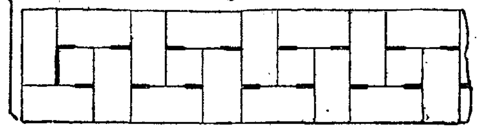


Fig. 148.

Figs. 147 and 148.—Courses for Double Flemish Bond.

Quarter, Half, and Three-quarter Bonds.

The term quarter-bond in brickwork applies equally to English and Flemish bond, but not to stretching bond, which latter might be called half-bond, as the bricks overlap half their length, but these special terms should be discountenanced; one name for a thing is sufficient. Three-quarter bond may refer to the use of three-quarter bricks instead of closers at the end of a wall in English or Flemish bond. A wall of more than half a brick thickness cannot be properly built in stretching bond without using iron ties.

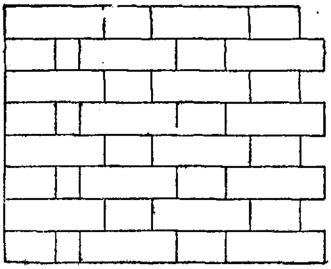


Fig. 149.—Elevation of Wall in Flemish Bond.

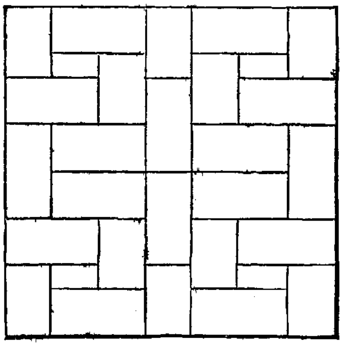


Fig. 150.

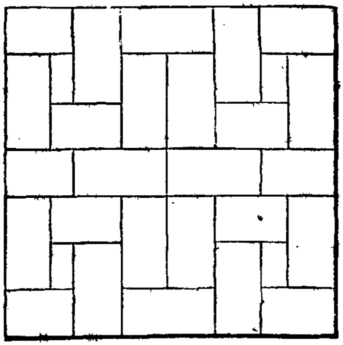


Fig. 151.

Figs. 150 and 151.—Pier in Flemish Bond.

Brick Pier in Flemish Bond.

A three-and-a-half brick pier in Flemish bond is a difficult one to bond satisfactorily, but a suggestion is given in Figs. 150 and 151. Figs. 152 and 153 show the bonding in alternate courses of a two-and-a-half brick pier. Figs. 154 and 155 show the elevation and plan of footings for this pier. The courses are laid as nearly as possible all headers, taking opposite directions in alternate courses.

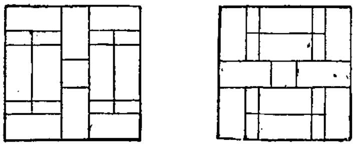


Fig. 152. Fig. 153.

Figs. 152 and 153.—Two-and-a-half Brick Pier in Flemish Bond.

English Garden Bond.

English garden bond has generally one course of headers to four or five courses of stretchers, because of the difficulty in getting the headers of the right length to go through a 9-in. wall and be flush on both sides. English garden bond may also consist of one course of headers to three of stretchers, the heading courses containing a closer next but one to the end. There is no hard and fast rule as to this bond; such walls are, in fact, built in all kinds of bonds. A bond customary in the North Country is shown in Fig. 156. The great point is not to have one heading brick directly over another, and this object is attained by

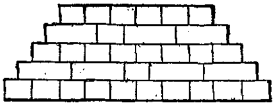


Fig. 154.

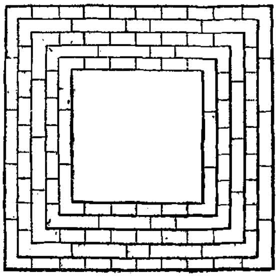


Fig. 155.

Figs. 154 and 155.—Footings for Pier in Flemish Bond.

fixing a three-quarter brick next the quoin in the top heading course, then quarter end next quoin in centre heading course; then, in bottom heading course, fix stretcher and quarter end, then the header will follow in the ordinary way, working three stretching to one heading course. The writer believes this to be a very good bond, but it would be improved by shifting the closer, in the second course from the bottom, to the other side of the stretcher. For a very usual English garden bond see Fig. 157. Flemish garden bond or Sussex bond is shown by Fig. 158.

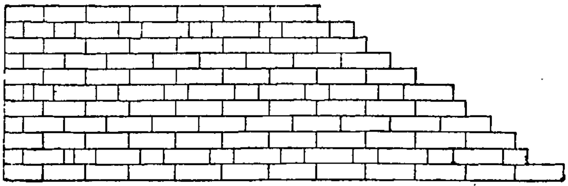


Fig. 156.—Mixed Garden Bond. (North Country.)

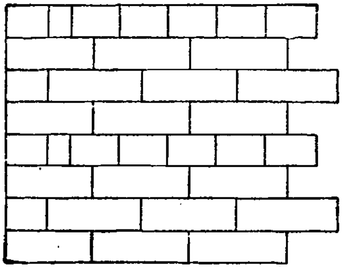


Fig. 157.—English Garden Bond.

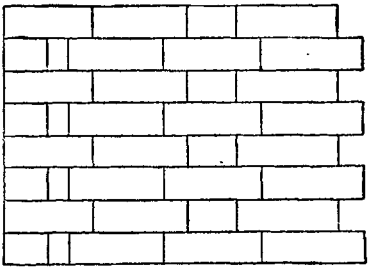


Fig. 158.—Flemish Garden Bond or Sussex Bond.

Some Other Bonds.

Stretching bond or chimney bond for half-brick walls is shown by Fig. 159; heading bond used for circular courses by Fig. 160; St. Andrew's Cross bond or English Cross bond (header inserted near end of alternate stretching courses) by Fig. 161; broken bond between windows by Fig. 162; and diaper bond for panels in half-timbered work by Fig. 163.

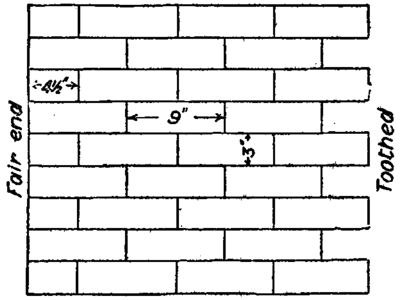


Fig. 159.—Stretching Bond or Chimney Bond.

Ranging Bond.

Ranging bond consists of narrow horizontal pieces of wood built into the joints of the walling parallel to one another, at intervals of about 18 in., in order to form grounds for battening, etc. (see Fig. 164). The faces of the pieces project slightly from the wall, so that the battens may be clear of the masonry.

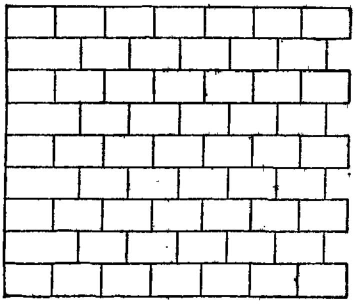


Fig. 160.—Heading Bond.

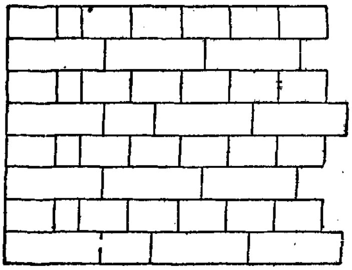


Fig. 161.—St. Andrew's Cross Bond or English Cross Bond.

Diagonal and Herring-bone Bond.

The method of laying out a 4-in. brick wall in diagonal bond with English facing is shown in Fig. 165. Draw the facing bricks first, then take the diagonal length of four bricks A to B, strike an arc B C; from C with half-brick radius, strike arc D E, join E A, and from E, the point of intersection with B C, draw horizontal line E F. From F draw F G parallel with E A, to form one of the diagonal joints; all the others may be obtained by drawing parallel lines at half-brick intervals. This method enables the diagonal bricks to be laid without any cutting, but it may be preferred to fill in solid the triangular end spaces H I J, etc.; then the diagonal joints would be laid at 45°, and the bricks cut to fit. Every alternate course must be all headers, as shown in Fig. 166, and the diagonal courses may rake alternately to the right and the left. Fig. 167 shows herring-bone bond, the interior bricks being laid at 45° and cut to fit the face bricks. Zig-zag is shown by Fig. 168. This is used for panels only, similarly to Fig. 163, as it can carry no weight.

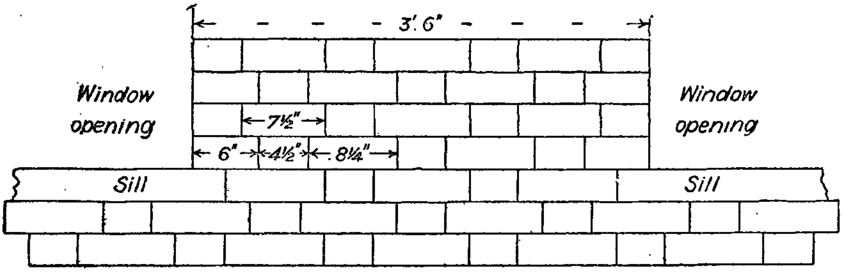


Fig. 162.—Broken Bond between Windows.

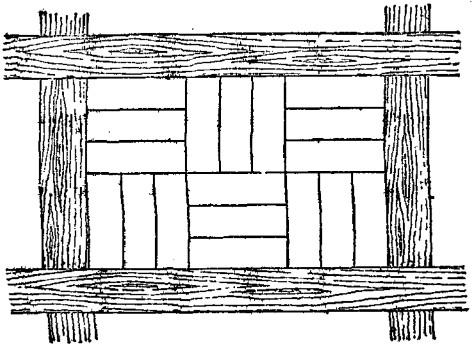


Fig. 163.—Diaper Bond for Panels in Half-timber Work.

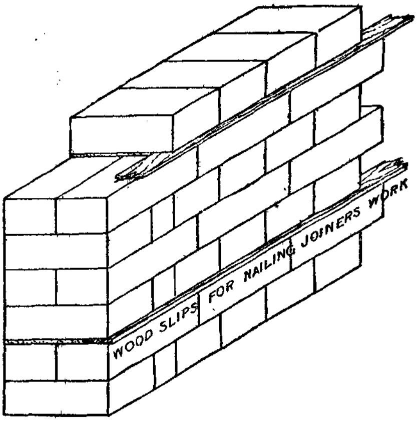


Fig. 164.—Ranging Bond.

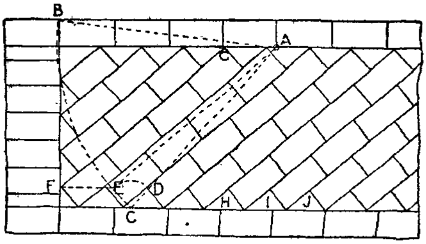


Fig. 165.

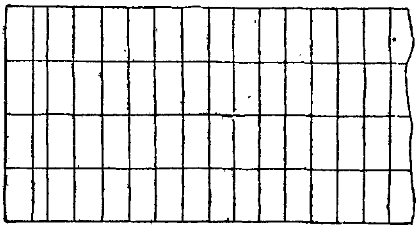


Fig. 166.

Figs. 165 and 166.—Diagonal Bond.

Some Terms Explained.

Perpends is the name given to the vertical joints in the face of brickwork ; "keeping the perpends" means keeping the vertical joints truly in line.

Reveals are the exposed portions of brickwork or stonework at the sides of door and window openings at right angles to the face work.

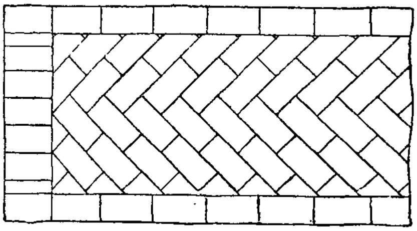


Fig. 167.—Herring-bone Bond.

Dinging, as a term applied to brickwork, has various significations ; it is used for the rubbing of brick facings with a soft brick of the same colour, in order to give uniformity of tint before pointing ; the term is also used for the axing of old brick facings with a bricklayer's axe in order to give a new face before re-pointing. And, again, it is the smoothing of the face preparatory to limewhiting.

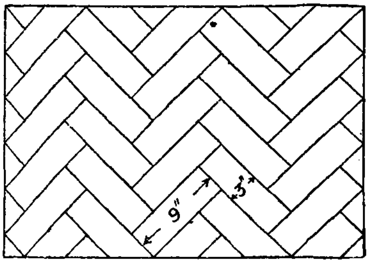


Fig. 168.—Zig-zag Bond.

Jambs are the portions of brickwork or stonework at the sides of openings for doors, windows, fireplaces, etc.

Grouting consists in using thin or liquid mortar, or cement when building is in cement mortar, poured between the bricks to fill up completely any spaces not properly filled up during the laying of the bricks. It is applied over every second or third course of brickwork in thick walls and foundations to fill up empty joints left through careless or hurried workmanship. It must not be at greater intervals than every fourth course, as otherwise the weight of the grouting may burst the mortar joints, besides encouraging slovenly and careless work. Some architects prefer larrying to grouting.

Larrying is the use of very soft semi-liquid mortar laid on the work and pushed up in the vertical joints by sliding the bricks along. Larrying is often adopted in thick walls after the face bricks are laid, and then consists in spreading soft mortar in a thick layer over the bed, and putting each brick down some distance from its ultimate position and pushing it into that position, thus filling up the joints between the bricks. This, again, is not to be advocated, as all mortar should be used as stiff as possible. Its object is to get through the work quickly.

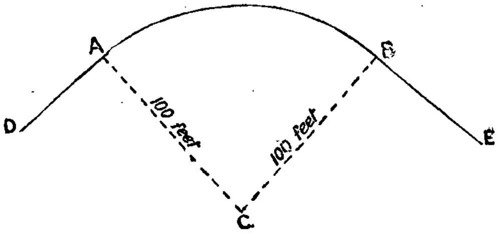


Fig. 169.—Setting Out Curve with Tape and Peg.

Setting Out Curve for Wall.

The handiest way to set out a curve of 100 ft. radius is to use a 100-ft. steel tape, and swing it on a peg at the centre. Failing this, a piece of whipcord, such as is used on a chalk reel, will answer. If the points A and B (Fig. 169) represent the extremities of the curve, arcs to intersect at C should be struck from these points, and C will then be the centre for the curve. The pieces of straight line A D, B E should be tangent to the curve, then D A C and E B C will be right angles.

Setting Out by Chords and Offsets.

For a second case, assume that a wall about 500 yd. long is to be built on slightly sloping ground, with about half the length straight and the remainder curved, the curve to be fixed by trial. Owing to the distance and the unequal nature of the ground a centre cannot be obtained; and so the method adopted will differ from that just given. The best method of setting out the curve will be by means of chords and offsets, as shown in Fig. 170, where A B is the straight part of the wall, the curved part commencing at B. Peg a chain, or line, down at B, make B C a given distance in the same straight line with A B, say 50 ft. or 1 chain, and swing the end C through an arc C D that will give point D in the required curve. Now peg down at D the end that was at B, sight through B D to fix point E in the right direction, D E being equal to B D, and swing end E through arc E F so that E F is twice C D, both being measured straight from point to point. Then B D F are points in the curve, and any number of succeeding points may be obtained by sighting through the two previous ones and taking the same distance and same offset, as all offsets after the first are equal so long as the curve continues of same radius. When a sharper curve is desired the only alteration is to swing the line through a longer offset.

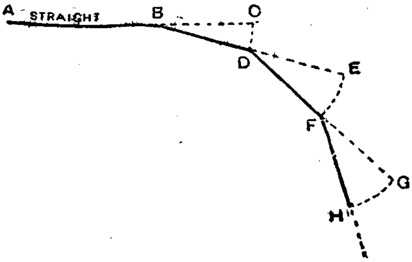


Fig. 170.—Setting Out Curve by Chords and Offsets.

Formula for Setting Out Template Curve.

Supposing that a wall is being built in which occurs a curve of 140-ft. radius. A part of this curve can be set out on a template, say, 9 ft. or 10 ft. in length. In Fig. 171 let $l =$ the length of template required, $r =$ radius of curve, $x =$ rise or versine of curve, $m =$ chord of half the arc, $y =$ rise or versine of the chord of half arc.

$$x = r - \sqrt{r^2 - \left(\frac{l}{2}\right)^2}$$
$$= 140 - \sqrt{19{,}600 - 25}$$
$$= \cdot 0893 \text{ ft., say } 1\tfrac{1}{16} \text{ in.}$$

$$m = \sqrt{\left(\frac{l}{2}\right)^2 + x^2}$$
$$= \sqrt{25 + \cdot 0893^2}$$
$$= 5\cdot 0008$$
$$y = r - \sqrt{r^2 - \left(\frac{m}{2}\right)^2}$$
$$= 140 - \sqrt{140^2 - 2\cdot 5004^2}$$
$$= \cdot 02 \text{ ft., say } \tfrac{1}{4} \text{ in.}$$

Then with l, x, and two y's there will be five points given on the curve, practically fixing the shape of the template.

The Same Formula in Simple Arithmetic.

Put into non-mathematical language, the above method is as follows: Take the given radius and multiply it by itself, 140 × 140 = 19,600; from this subtract half the length of template also multiplied by itself; half 10 equals 5, and 5 multiplied by 5 equals 25, 25 taken from 19,600 leaves 19,575. Now what is called the square root of 19,575 is wanted, that is, a number which, multiplied by itself, shall equal this quantity. Without a knowledge of square root it will take some time to get this, but ultimately it will be found to be 139·91. Take this from the given radius, 140 − 139·91 = ·09 ft., multiply this by 12, which gives 1·08 in., or say, $1\frac{1}{16}$ in. full. Now take a piece of stuff 7 in. × 1 in., plane one face and shoot one edge, cut it off to 10 ft. 6 in. in length, gauge a line 3 in. from the true edge, square a line across the centre of it, and two others exactly 5 ft. away, then gauge in the centre a short line $4\frac{1}{16}$ in. full from the edge; it is then ready for marking out the curve. Fig. 172 shows the piece of wood with the gauge lines on it, and the lines now to be described. From the intersection of the gauged line and squared line at A, draw a line by a straightedge through the intersection of the short gauge line and squared line B, to meet the squared line C.

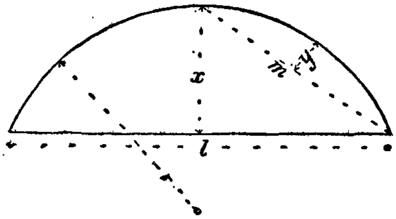


Fig. 171.—Setting Out Template Curve.

Then C D should be exactly double B E. Divide D E into five equal parts, which will be at every foot, and divide C D into the same number of equal parts somewhat less than ½ in. each. Square lines across through 1, 2, 3, 4, on D E, and then draw lines, or parts of lines, from A to 1, 2, 3, 4, on C D, cutting the lines D E. Where the lines from similar numbers cut each other will be points in the curve, and the other half of the template may be set out in the same way. Then cut out and trim up to F D B A G, or use the upper half C D B A H if a hollowed template is required.

Ensuring Accuracy of Bricklayers' Work.

Suppose that the bricklayer is starting level, he first builds up two piers, one at each end of the wall to be built, raking the brickwork back at each course. These piers, which he takes up six or eight courses, he builds so that any four courses equal 1 ft. in height, or perhaps so that any four courses exceed in height by 1 in. the same bricks laid dry. He builds them true with each other by stretching a line between them, fixing the same in the piers by means of line-pins stuck in the mortar joints. As the piers are built up, he uses his plumb-rule to test their perpendicularity. He should also check every four courses or so, to see that his work is accurately horizontal. This is done either with a long spirit level, levelling the whole length of the wall at once, or by means of a straightedge and spirit level, levelling a portion equal to the length of the straightedge used. Having now built up his piers in a line with each other, the courses being horizontal and the face upright, he fills in the intermediate spaces, laying one course at a time right through from pier to pier, working to his line fixed in the piers as before described, which he raises one course at a time as it is necessary. These principles will apply to any kind of work which he may be required to do.

Bricklayers' Plumb-rule.

This rule consists of a piece of yellow deal or pine about 4 ft. or 5 ft. long, 4 in. or 4½ in. wide, and ½ in. thick, its edges being planed perfectly straight and parallel to each other, and having a scribed line cut down the middle of its width, at one end of which, about 4 in. from the end of the rule, a hole is cut, into which the lead bob will easily swing when suspended from the top of the rule by means of a string. By placing one edge of this rule against the piers, the bricklayer knows that his work is upright if the string and bob are in the middle of the rule.

Bricklaying in Frosty Weather.

There is no recognised time or period of the year at which bricklaying should be suspended on account of frost; this is a matter that is determined by the weather and by local custom. Generally speaking, in England work continues throughout the winter, the only protection being a scaffold board, laid on the top course; but sometimes sacking is laid over the upper courses when the work is left for the night. If a hard frost sets in, the work may be suspended until the frost breaks; but in Sweden and Norway building operations are not so readily interrupted, as sugar is added to the mortar in order to lessen the liability to freezing. In the United States and Canada brickwork in cement mortar is continued in frosty weather by using hot water for mixing the mortar.

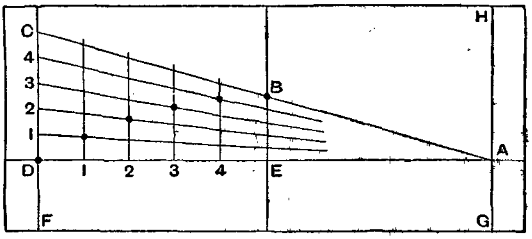


Fig. 172.—Setting Out Curve Template.

Mortar Joints in Brickwork.

There is nothing to be said in favour of thick joints considered only as such in modern or any other brickwork. If the bricks are irregular they require thicker joints to obtain an even bed; but unless the mortar is of at least the same strength as the bricks, the thinner the joint the more substantial the work.

Efflorescence on Bricks.

New brickwork often shows a flocculent white substance on the face, which may last for some time, but ultimately disappears. There may be various causes, but it is generally attributed to salt in the bricks, or the use of sea sand in the mortar. If it is very unsightly, a bass broom might be used to remove it.

Re-using Old Bricks.

Old bricks, if sound, hard, and dry, make thoroughly good work for plastering over, but care must be taken to brush the bricks down and well sprinkle them with water just before the plaster is laid on, as old bricks are apt to be rather dusty and absorbent.

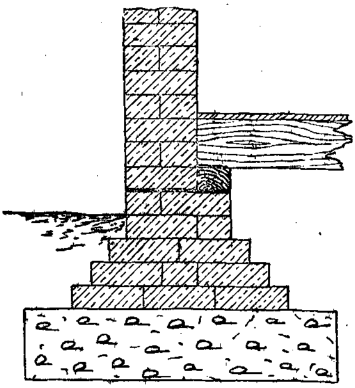


Fig. 173.—Slate Damp-proof Course.

Material for Setting Gauged Brickwork.

Fine gauged brickwork is sometimes set in mastic composed of powdered brick (of the same kind as the brickwork) mixed with boiled linseed oil and litharge, or in plasterers' putty composed of pure lime slaked to a cream and run through a fine sieve. Brickwork for carving should be set in shellac varnish and whitelead ground in oil, well mixed together; or in patent knotting and whitelead mixed to a creamy paste. Gauged work generally is set in bricklayers' putty, which is simply good mortar without lumps, reduced to a cream by the addition of water. Masons' putty for stonework is formed of lime, whitelead, and a little fine washed silver sand or marble dust.

Waterproofing Brick Wall.

The various preservatives for stone or brick are mostly silicates of potash or soda. Szerelmey's stone liquid and the Bath Stone Firms' Fluate are both well known, and are appreciated. The solution is generally applied with a whitewash brush, and brushed over until, although dry to the touch, the solution still glistens as if wet. A solution of calcium chloride is brushed on after the silicate is perfectly dry; this causes a double decomposition, in which insoluble silicate of lime is formed.

Loads on Brickwork.

The following list shows the approximate loads at which failure commenced in the tests carried out in 1896 by a Committee of the Royal Institute of British Architects at the West India Docks, London:—Stock bricks in mortar, 5 tons per square foot; stock bricks in cement, 7½ tons per square foot. Gault bricks in mortar, 10 tons per square foot; Gault bricks in cement, 20 tons per square foot; Leicester red bricks in mortar, 30 tons per square foot; Staffordshire blue bricks in mortar, 35 tons per square foot; Staffordshire blue bricks in cement, 50 tons per square foot.

Damp-proof Courses.

The usual kinds of damp-proof courses for external walls are:—(a) A course of slates throughout the thickness of the wall, 3 in. to 6 in. above ground line. (See Fig. 173.) (b) A layer of asphalt from the level of the damp-proof course to above ground line. In Fig. 174 the asphalt covers both faces as well as the footings. (c) Glazed and perforated stoneware

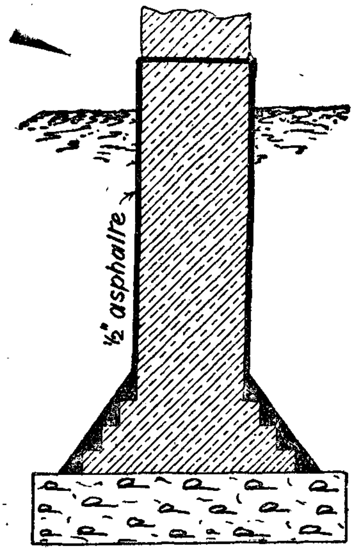


Fig. 174.—Asphalt Damp-proofing, Horizontal and Vertical.

slabs, of which a variety is shown by Figs. 175 to 181. (*d*) A layer of melted pitch with sufficient coal tar mixed with it to prevent it setting too brittle. (*e*) A layer of sheet lead, 4 lb. to 8 lb. per superficial foot, with 1½ in. laps. This is perhaps the best damp-proof course if placed upon a thin layer of Portland cement, and covered in the same way; it is then practically unaffected by the lime, though in contact with damp, porous mortar it would in time be converted to carbonate of lead. (*f*) A layer of asphalted (that is, tarred) roofing felt laid dry.

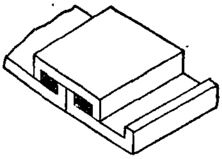


Fig. 175.—Taylor's Original Damp-proof Course.

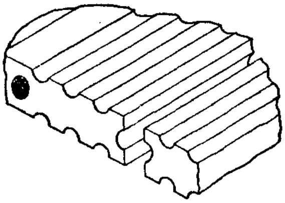


Fig. 176.—Taylor's Modern 1-in. Damp-proof Course.

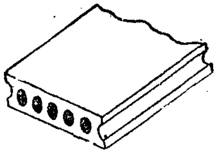


Fig. 177.—Taylor's 1½-in. Damp-proof Course.

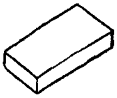


Fig. 178.

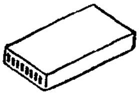


Fig. 179.

Figs. 178 and 179.—Taylor's Damp-proof Tiles to be laid alternately.

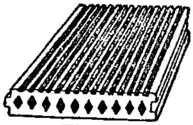


Fig. 180.

Fig. 180.—Doulton's Damp-proof Course.

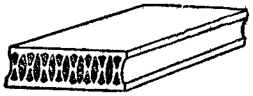


Fig. 181.

Fig. 181.—Jennings' Damp-proof Course.

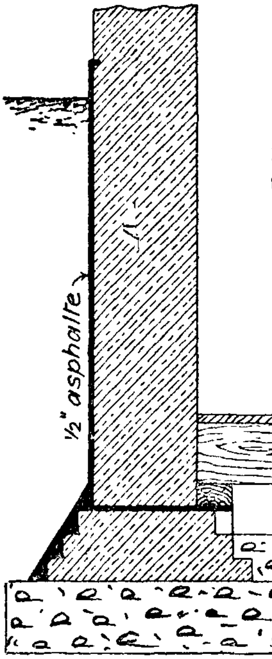


Fig. 182.—Vertical Asphalt Damp-proofing.

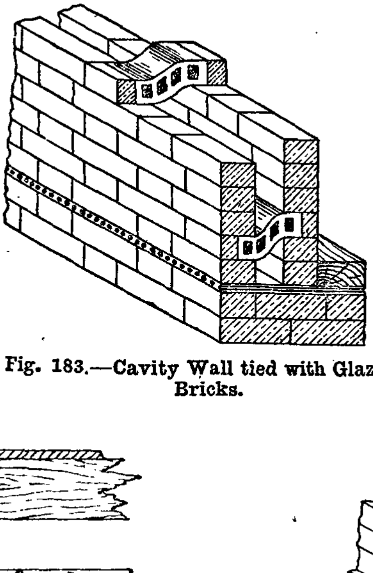


Fig. 183.—Cavity Wall tied with Glazed Bricks.

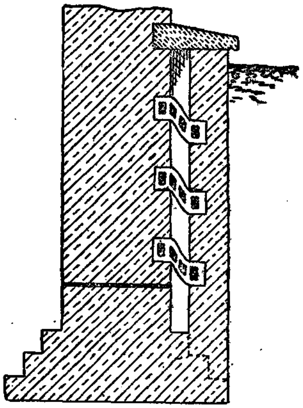


Fig. 184.—Cavity Wall tied with Glazed Bricks.

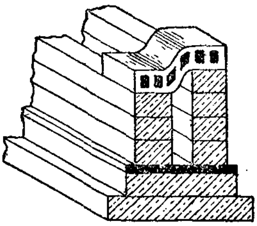


Fig. 185.—Cavity Wall tied with Glazed Bricks

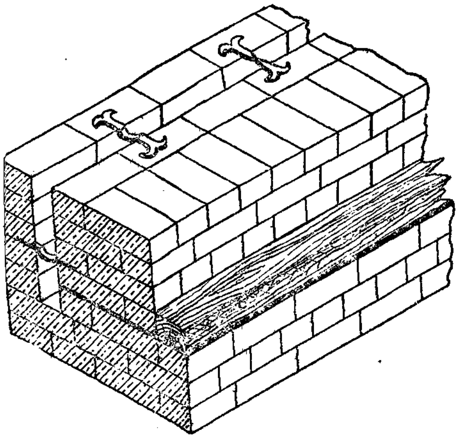


Fig. 186.

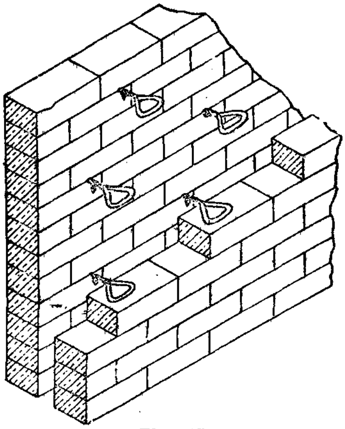


Fig. 187.

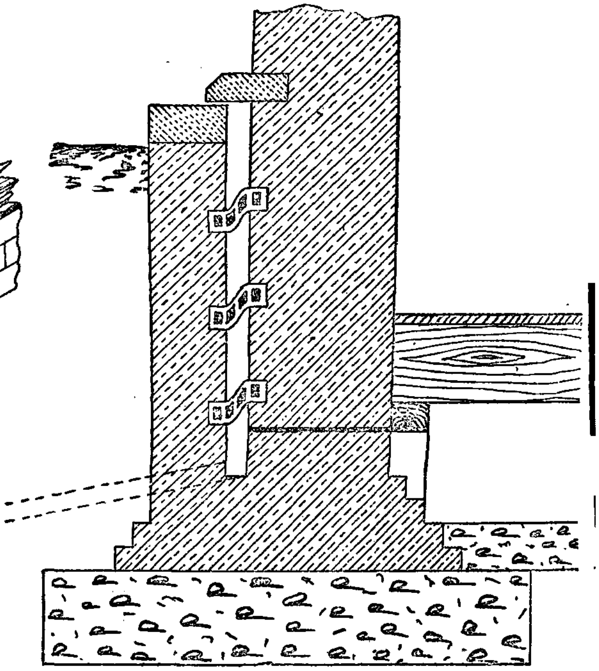


Fig. 191.

Fig. 186.—Modern Cavity Wall with Galvanised Iron Ties. Figs. 187 and 188.—Warrington Bond Iron in Cavity Wall. Figs. 189 and 190.—Jennings' Stoneware Bonding Bricks or Wall Ties in Cavity Wall. Fig. 191.—Cavity Wall with Stone Capping. Fig. 192.—Hygeian Rock Damp-proofing.

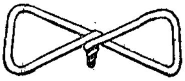


Fig. 188.

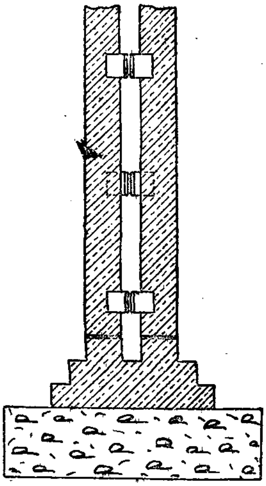


Fig. 189.

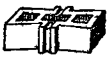


Fig. 190.

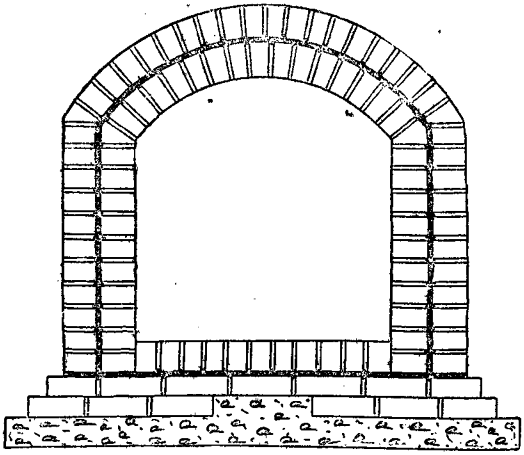


Fig. 192.

Vertical Damp-proofing.

To prevent the damp coming through the vertical face of a wall (*a*) it may be covered with asphalt from the level of the damp-proof course to above ground line, or as in Fig. 182 or Fig. 174. (*b*) It may be coated with hot pitch. (*c*) It may be rendered in Portland or Roman cement, ½ in. to ¾ in. thick. (*d*) It may be built with a 2-in. cavity and tied with glazed bricks or galvanised iron ties. (See Figs. 184 to 191.) (*e*) The space may be grouted with Hygeian or Tenax rock composition. (See Figs. 192 and 193.) (*f*) The inside may be covered with thin sheet lead, or, if plastered, may be hung with tinfoil paper. (*g*) The outside may be covered with slates or hanging tiles; Fig. 194 shows Taylor's vitrified stoneware facing. Tile creasing and stone copings as in Figs. 195 and 196, and drip tiles and slate creasing as in Figs. 197 and 198, are also useful. Fig. 199 shows a form of vertical and horizontal damp-proofing consisting of a ½-inch thickness of asphalt.

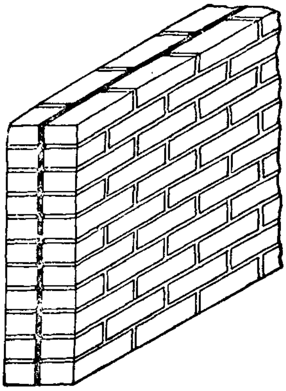


Fig. 193.—Tenax Rock Vertical Damp-proofing.

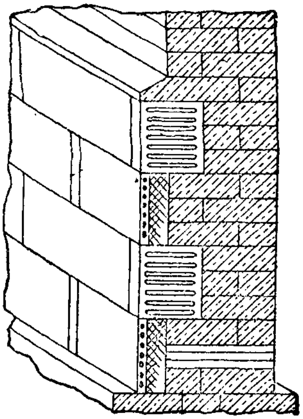


Fig. 194.—Taylor's Vitrified Stoneware Facing.

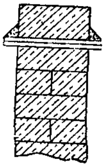


Fig. 195.—Tile Creasing.

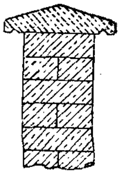


Fig. 196.—Stone Coping.

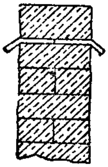


Fig. 197.—Drip Tiles.

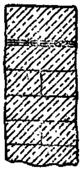


Fig. 198.—Slate Creasing.

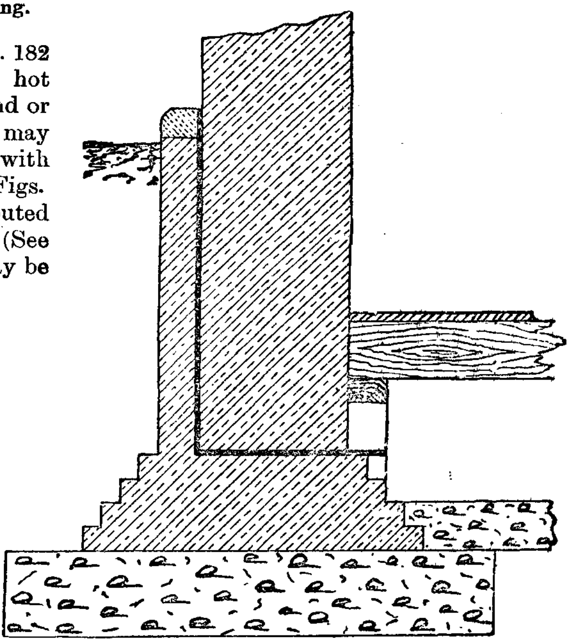


Fig. 199.—Vertical and Horizontal Damp-proofing.

Hollow or Cavity Walls.

The custom in building hollow or cavity walls is to put a skin of 4½-in. brickwork outside the ordinary thickness that may be required, according to the height of the building, length of wall, load on floors, etc. The 4½-in. thickness is placed at a distance of 2 in. from the main wall. The joists and roof timbers are then carried in the ordinary way, and the safety of the structure is not endangered. The outer skin is connected to the main wall by means of galvanised iron ties (Figs. 200 and 201), about one to every 4 ft. super.; the gauged arches are formed in it; the lintels are in the main wall only; sheet-lead is built in the outer wall over door and window frames, turned up at the back, and running about 2 in. beyond each end. The bottom course of the outer wall is not less than one course below the damp-proof course of the inner wall, and has air bricks in it at intervals of 4 ft. to 6 ft., to allow of a constant upward current of air to dry out any moisture; similar air-bricks are put at the top if the hollow space is covered. Cavity walls can hardly be said to be much used, when the number of buildings erected with solid walls is taken into account; but for detached country residences of two floors cavity walls are fairly frequent. It will be noted that the 4½-in. wall is in addition to the ordinary thickness; it would not answer to have two 4½-in. walls 2 in. apart, connected by wall ties; these would certainly not be so strong as one 9-in. wall.

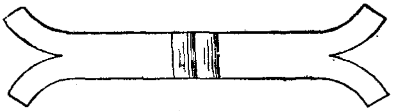


Fig. 200.

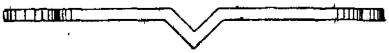


Fig. 201.

Figs. 200 and 201.—Galvanised Iron Ties.

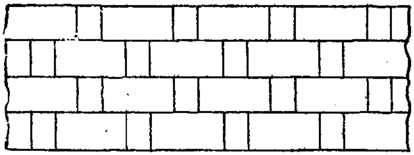


Fig. 202.

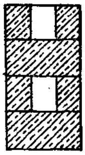


Fig. 203.

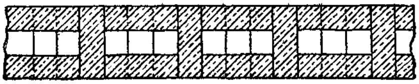


Fig. 204.

Figs. 202 to 204.—Silverlock's Hollow Wall.

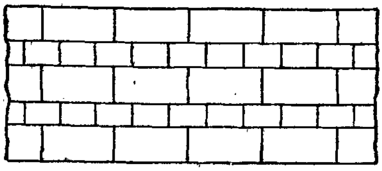


Fig. 205.

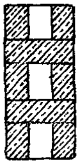


Fig. 206.

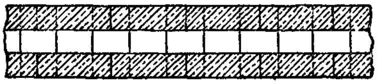


Fig. 207.

Figs. 205 to 207.—Dearne's Hollow Wall.

Types of Hollow Walls.

In other paragraphs technical details of hollow or cavity walls are given. The present purpose is to draw attention to some explanatory figures. Figs. 184 to 189, showing such walls, have already been alluded to. Silverlock's 9-in. wall is illustrated by Figs. 202 to 204; Dearne's 9-in. wall by Figs. 205 to 207; Loudon's 11-in. wall by Figs. 208 to 210; and Allen's 12-in. wall by Figs. 211 to 213. In Figs. 214 and 215 an air drain is

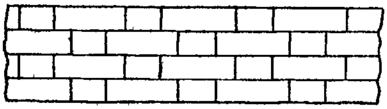


Fig. 208.

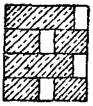


Fig. 209.

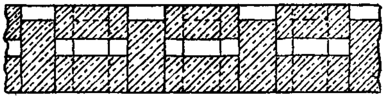


Fig. 210.

Figs. 208 to 210.—Loudon's Hollow Wall.

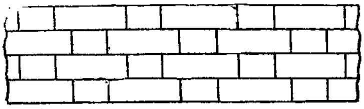


Fig. 211.

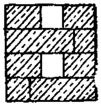


Fig. 212

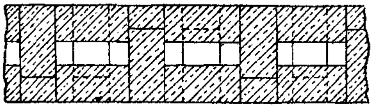


Fig. 213.

Figs. 211 to 213.—Allen's Hollow Wall.

shown, the footings in the latter figure being covered with asphalt, and wall ties being inserted across the cavity as shown. Fig. 216 represents the section of a closed dry area. An open area is shown by Fig. 217.

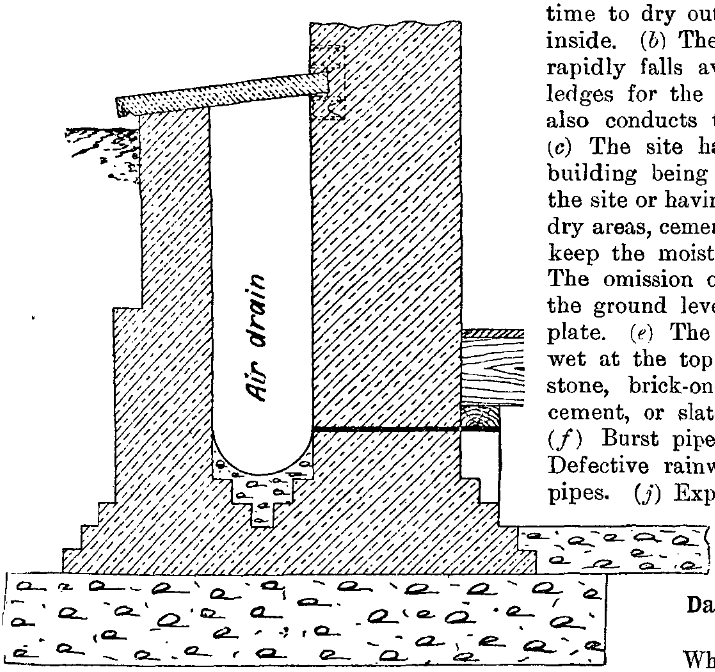


Fig. 214.—Air Drain with Stone Capping.

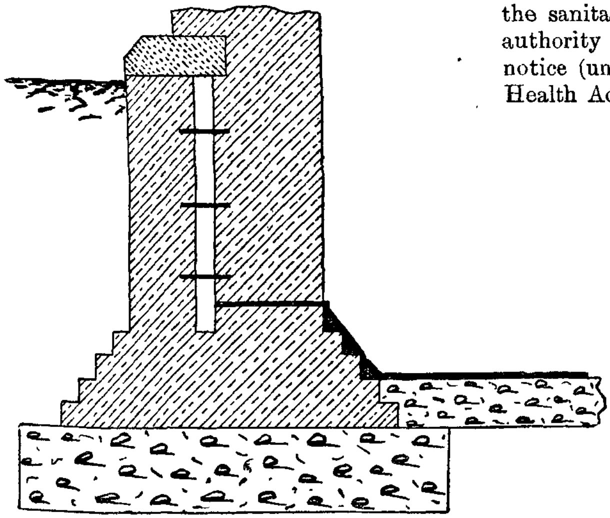


Fig. 215.—Air Drain with Wall Ties.

Causes of Damp Walls.

The chief causes of damp walls in houses are: (*a*) The use of common, underburnt bricks, which are porous and allow the rain and moisture to penetrate to such a depth that it takes a long time to dry out, and much of it reaches the inside. (*b*) The use of bad, soft mortar, which rapidly falls away at the joints, and leaves ledges for the rain to settle on; the mortar also conducts the moisture to the interior. (*c*) The site having a wet subsoil, and the building being erected without first draining the site or having some precaution taken, as by dry areas, cement or asphalt rendering, etc., to keep the moisture away from the walls. (*d*) The omission of a damp-proof course above the ground level and below the lowest wall-plate. (*e*) The absence of a protection from wet at the top of the walls—such as coping-stone, brick-on-edge and sailing course in cement, or slate or tile creasing in cement. (*f*) Burst pipes. (*g*) Defective gutters. (*h*) Defective rainwater pipe. (*i*) Defective soil pipes. (*j*) Exposed situation, especially near the sea. (*k*) House surrounded by large trees.

Damp Walls: Official Notice to Occupier.

When damp walls are such as to be injurious to health — and practically every damp wall is— the sanitary inspector acting under any local authority is empowered to give an official notice (under Sec. 91, Subsec. 1 of the Public Health Act, 1875, in the provinces, and Sec. 2

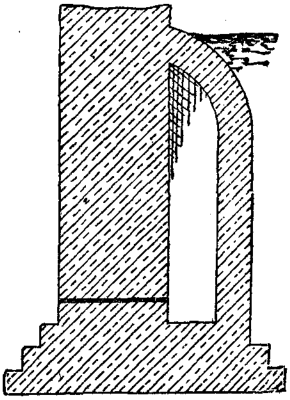


Fig. 216.—Closed Dry Area.

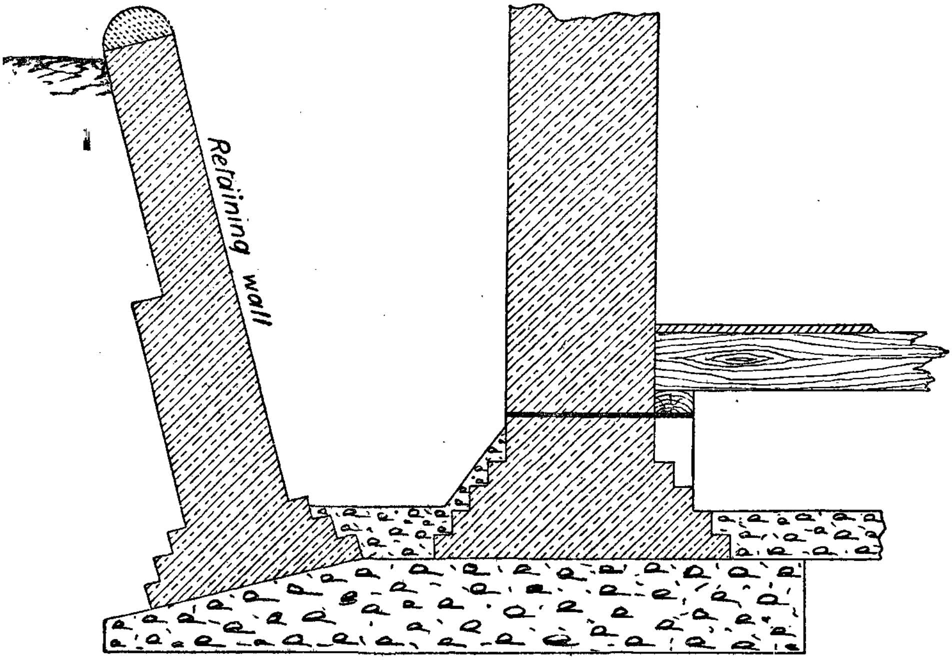

Retaining wall

Fig. 217.—Section of Dry Area.

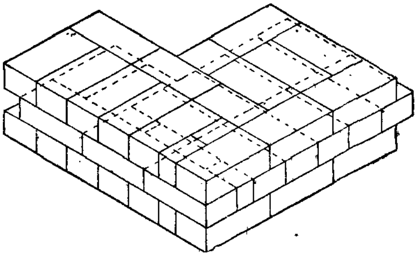


Fig. 218.

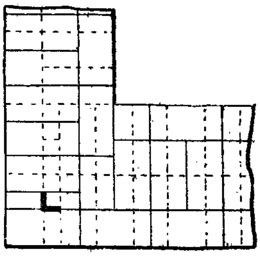


Fig. 219.

Figs. 218 and 219.—Angles in English Bond.

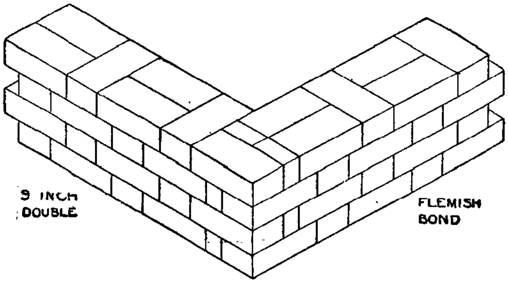

9 INCH
DOUBLE
FLEMISH
BOND

Fig. 220.

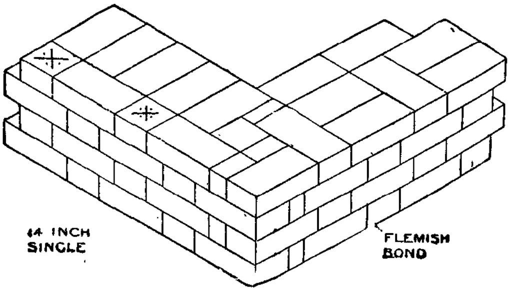

14 INCH
SINGLE
FLEMISH
BOND

Fig. 221.

Figs. 220 and 221.—Angles in Flemish Bond.

of the Public Health London Act, 1891, in London) to the occupier, setting forth the work required to be done to remedy the defects, and to summon him if it is not done, or to do the necessary work and recover the cost.

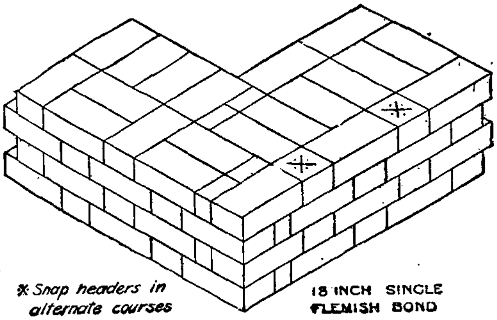

✕ Snap headers in alternate courses
18 INCH SINGLE FLEMISH BOND

Fig. 222.—Angle in Flemish Bond.

Angles in English Bond.

Fig. 218 is an isometric projection, to a scale of $\frac{1}{2}$ in. to a foot, showing how the bricks are laid at the angle of a building, the front wall being two bricks, and the end wall one-and-a-half bricks thick in English bond. The dotted lines in both Figs. 218 and 219 indicate the joints in the course below. Suitable mortar joints that would weather well in such a position are shown on pp. 120 and 121.

Angles in Flemish Bond.

Figs. 220 to 222 show respectively angles or quoins to 9-in., 14-in., and 18-in. walls in Flemish bond. Figs. 223 to 225 show the bonding for a 9-in. brick quoin in Flemish bond with two courses of footings. The bricks in the latter are laid, as far as possible, all headers. There will unavoidably be one straight joint near the angle between the three lower courses, but this is of little consequence.

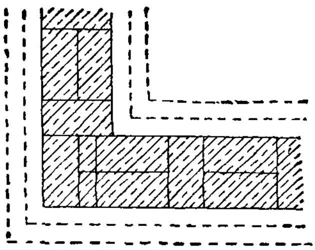


Fig. 223.—Angle in Flemish Bond.

Fig. 223 shows plan of the bottom course of wall, Fig. 224 upper course of footings, Fig. 225 lower course of footings.

Angle in Two-and-a-half Brick Walls.

In English bond the successive courses will be as in Fig. 226, and in single Flemish bond as in Fig. 227, in both of which figures the lower course is indicated by dotted lines. The thick lines show unbroken joints.

Salient Angle to be Rounded.

Salient angles are angles that form a projecting corner. A specification clause, "All salient angles internally where walls are not plastered to be rounded $2\frac{1}{2}$ in. radius; also externally, angles of back buildings, etc., etc.," means that quoin bricks are to be used for the projecting angles, the bonding being the same as if ordinary bricks with square ends had been used. The bonding at the end for glazed brick facings may be as shown in Figs. 228 and 229.

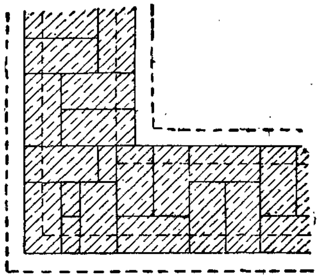


Fig. 224.

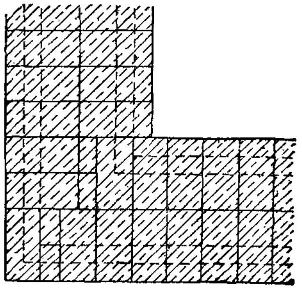


Fig. 225.

Figs. 224 and 225.—Footings for Angle in Flemish Bond.

Squint Quoins, etc.

A squint quoin (Fig. 230) is any external (or salient) vertical angle in brickwork or masonry that differs from a right angle. An internal or recessed vertical angle is called a birdsmouth.

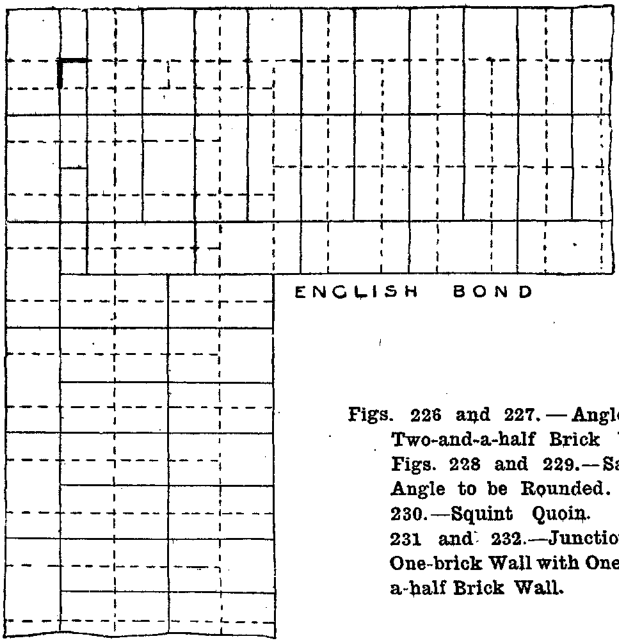

ENGLISH BOND

Fig. 226.

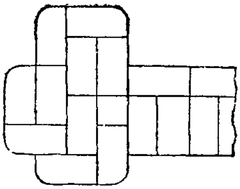


Fig. 228.

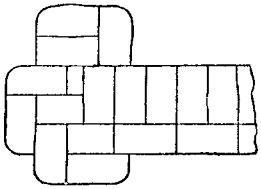


Fig. 229.

Figs. 226 and 227.—Angles in Two-and-a-half Brick Wall. Figs. 228 and 229.—Salient Angle to be Rounded. Fig. 230.—Squint Quoin. Figs. 231 and 232.—Junction of One-brick Wall with One-and-a-half Brick Wall.

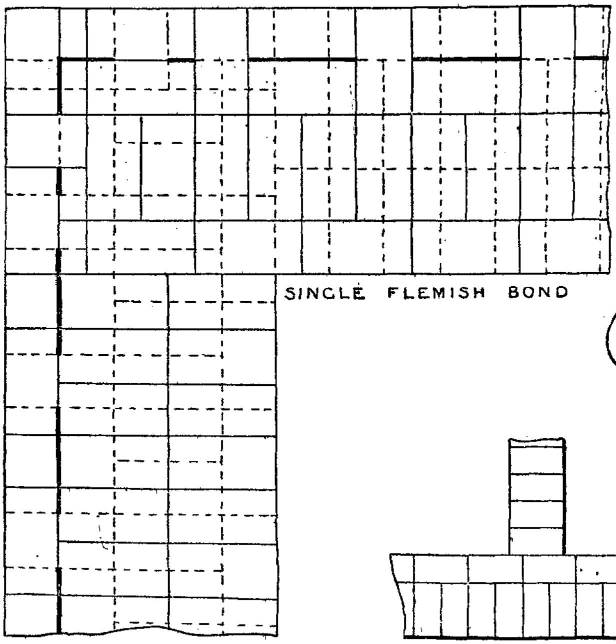

SINGLE FLEMISH BOND

Fig. 227.

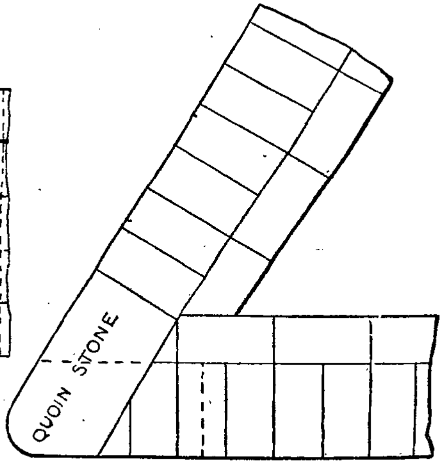

QUOIN STONE

Fig. 230.

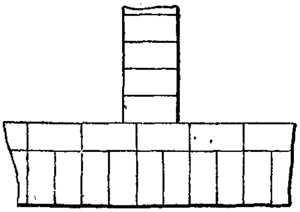


Fig. 231.

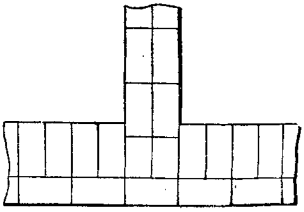


Fig. 232

Junctions in English Bond.

Figs. 231 and 232 show two successive courses of part of an external and cross wall in English bond. An alternative arrangement is presented by Figs. 233 and 234. Irregular work such as that shown by Figs. 235 and 236 can probably be bonded in several ways. The arrangement of the two courses given in Figs. 235 and 236 shows true bond and no straight joints inside the wall. A very peculiar case, probably necessitated by architectural requirements, is illustrated by Figs. 237 and 238. The bricks should be cut and properly bonded. Hoop-iron bond as shown by dotted lines will be a desirable addition, particularly if the walls are built in anything else but Portland cement mortar.

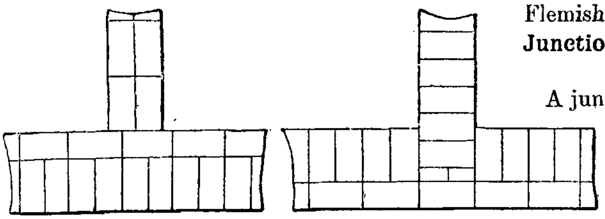


Fig. 233. Fig. 234.

Figs. 233 and 234.—Junction of One-brick Wall with One-and-a-half Brick Wall.

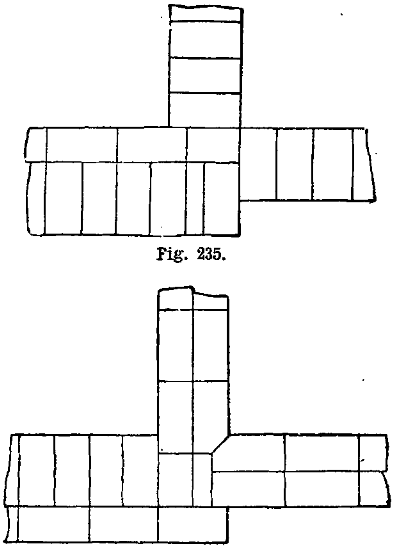


Fig. 235.

Fig. 236.

Figs. 235 and 236.—Irregular Junction.

Junction in Flemish Bond.

Two successive courses of brickwork at the junction of a party wall with the main wall of a building, the latter being built in single Flemish bond, are shown by Figs. 239 and 240.

Junction of Two-brick and Three-brick Walls.

A junction of a two-brick with a three-brick wall can be bonded in English style as in Figs. 241 and 242, or in Flemish style as in Figs. 243 and 244. The hatching in the latter figures indicates facing bricks.

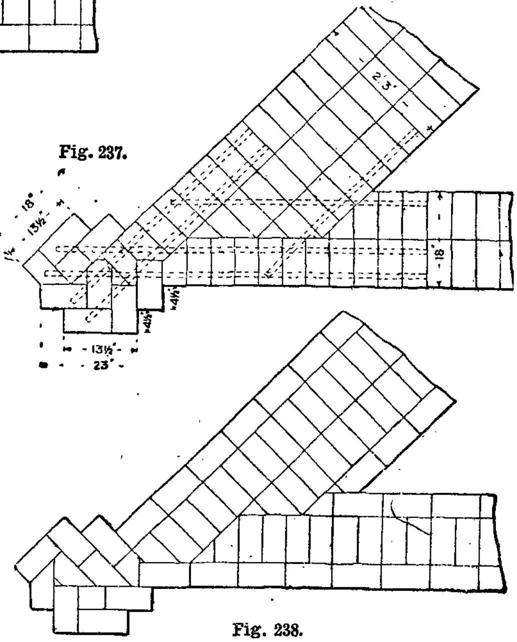


Fig. 237.

Fig. 238.

Figs. 237 and 238.—Peculiar Junction.

Bonding Brickwork at Openings.

A number of uniform illustrations has been prepared to show the bonding of brickwork at openings. It may be said that to allow face bricks to keep true bond the width of the opening should be any number of whole bricks for English bond, and two bricks plus any number of one and a half bricks for Flemish bond. The standard height of four courses of brickwork in London is 12 in., and in the Midlands slightly more than that. Successive courses in English, single Flemish and double Flemish bond respectively, suited to square

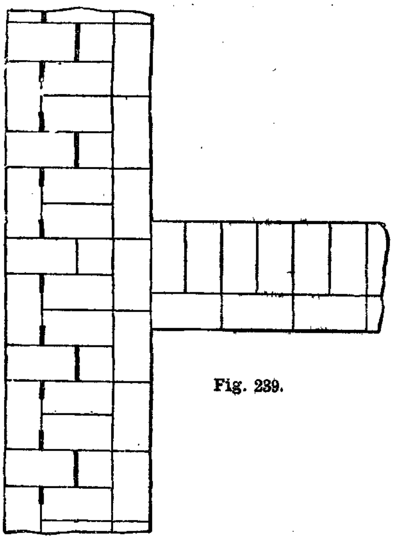


Fig. 239.

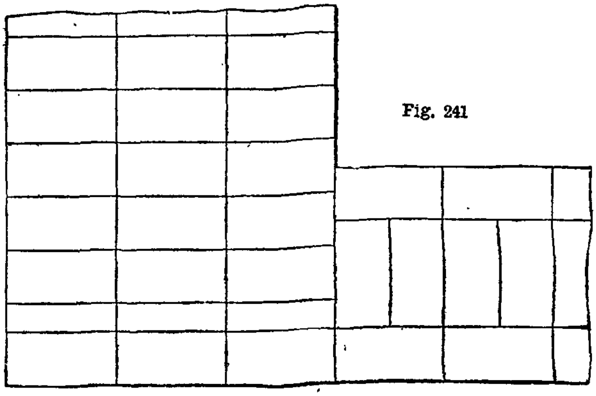


Fig. 241

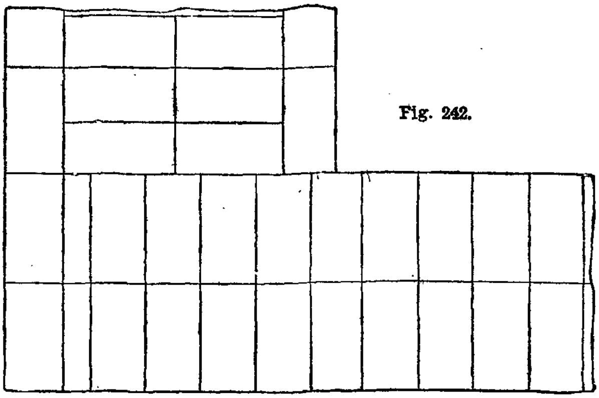


Fig. 242.

Figs. 241 and 242.—Angle of Two-brick Wall with Three-brick Wall (English Bond).

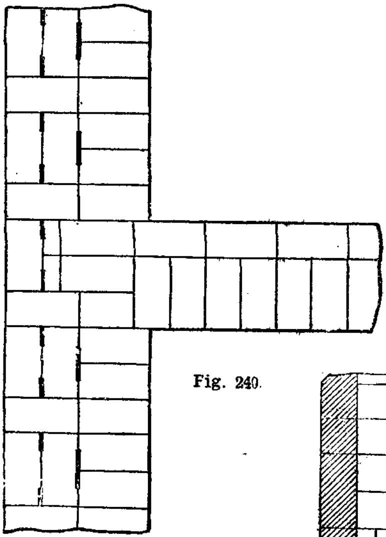


Fig. 240.

Figs. 239 and 240.—Junction between single Flemish Main Wall and English Party Wall.

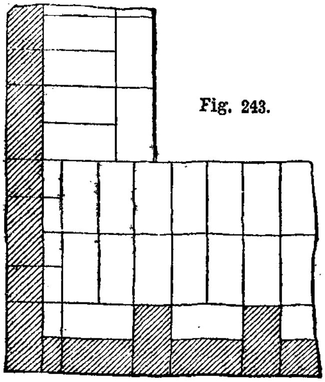


Fig. 243.

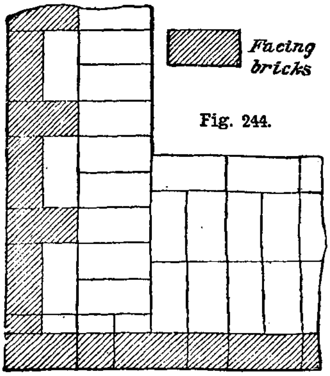


Fig. 244.

Figs. 243 and 244.—Angle of Two-brick Wall with Three-brick Wall, Flemish facing.

jambs, are illustrated by Figs. 245 to 253. Similarly, successive courses suited to recessed jambs are shown by Figs. 254 to 262. Loophole frames may be fixed within 1½-in. of the face of any external wall; but all other woodwork fixed in any external wall, except bressumers and the storey posts under them, and frames of doors and windows of shops on the ground storey of any building, shall be set back 4 in. at the least from the external face of such wall.

Brickwork to Outside Door Frame.

Fig. 263 shows a door frame for an outside door; it is set upon door-blocks. The illustration gives an elevation of the door frame, step, sill, and brickwork, and also the temporary bracing. Fig. 264 is an enlarged cross section of the frame to the left of Fig. 263, showing the plan of top course of brickwork. When there is an oak sill, as in this case, the door posts are usually fixed direct to the sill by stump tenons or ordinary tenons. Cast-iron sockets are sometimes used for the feet of door posts, and other

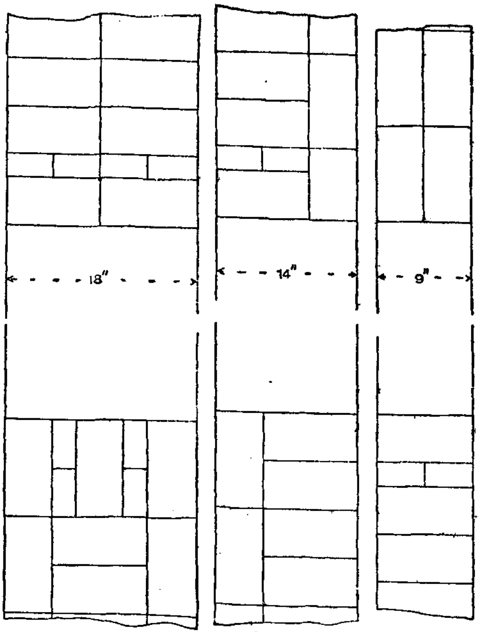


Fig. 245. Fig. 246. Fig. 247.

Figs. 245 to 247.—Openings in English Bond with Square Jambs.

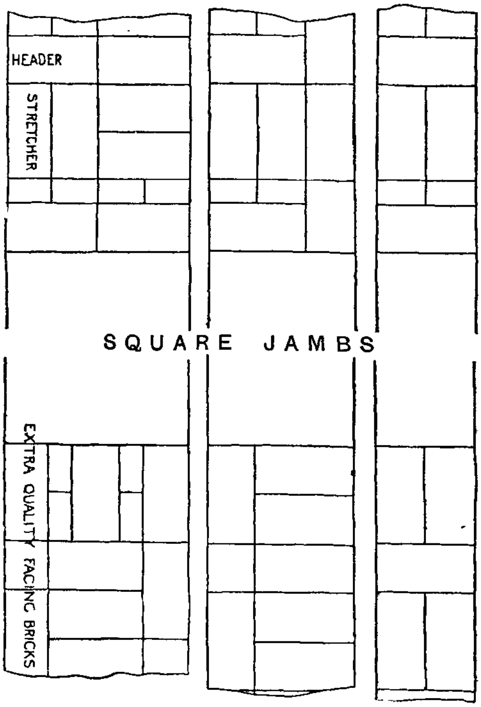


Fig. 248. Fig. 249. Fig. 250.

Figs. 248 to 250.—Openings in Single Flemish Bond with Square Jambs.

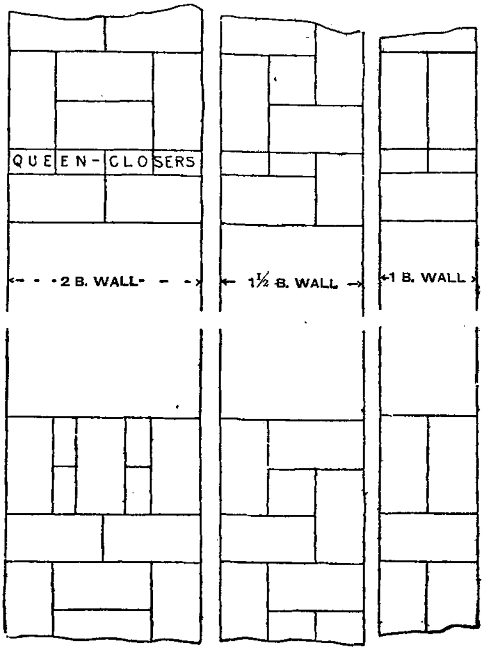


Fig. 251. Fig. 252. Fig. 253.

Figs 251 to 253.—Openings in Double Flemish Bond with Square Jambs.

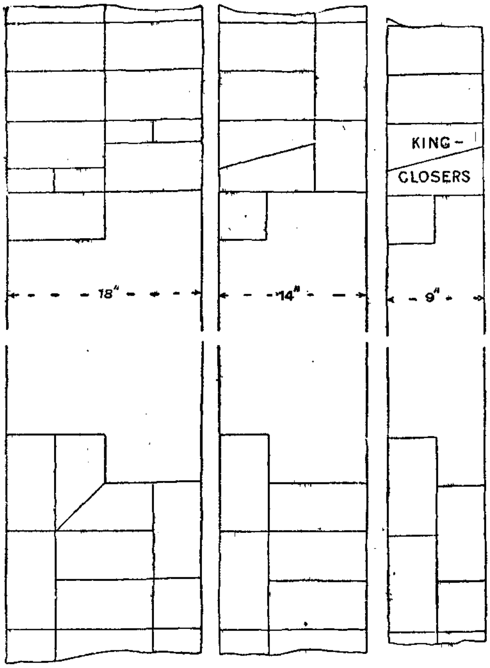

KING-
CLOSERS
18"
14"
9"

Fig. 254. Fig. 255. Fig. 256.

Figs. 254 to 256.—Openings in English Bond with Recessed Jambs.

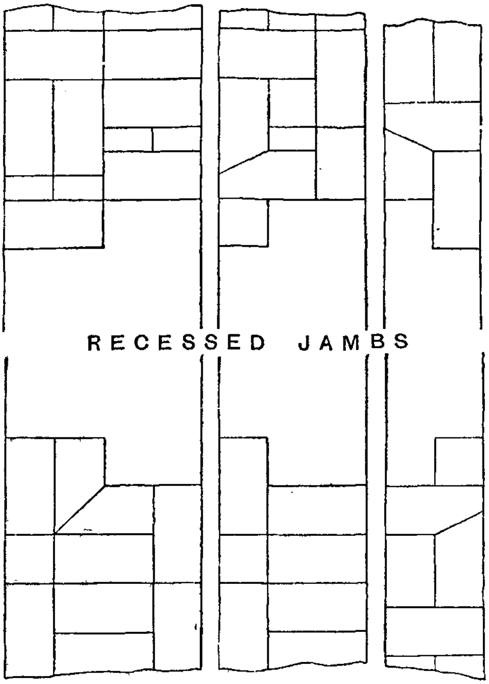

RECESSED JAMBS

Fig. 257. Fig. 258. Fig. 259.

Figs. 257 to 259.—Openings in Single Flemish Bond with Recessed Jambs.

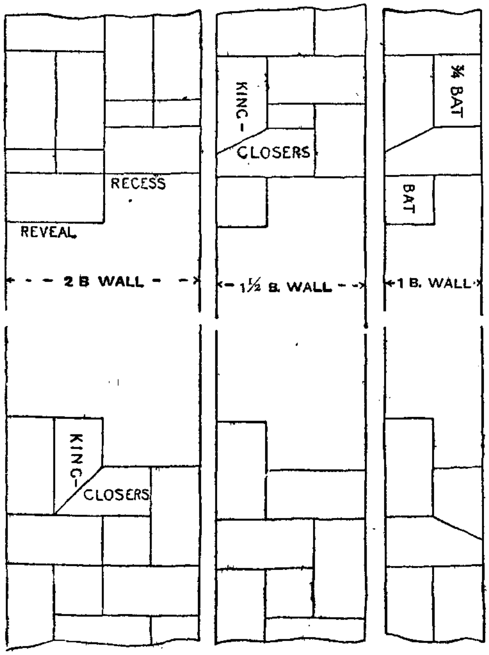

KING-
CLOSERS
¾ BAT
RECESS
REVEAL
BAT
2 B WALL
1½ B. WALL
1 B. WALL
KING-
CLOSERS

Fig. 260. Fig. 261. Fig. 262.

Figs. 260 to 262.—Openings in Double Flemish Bond with Recessed Jambs.

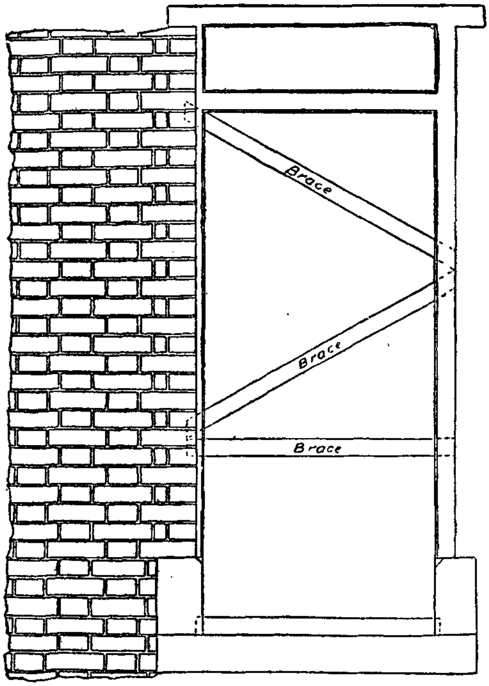

Brace
Brace
Brace

Fig. 263.—Doorframe and Brickwork.

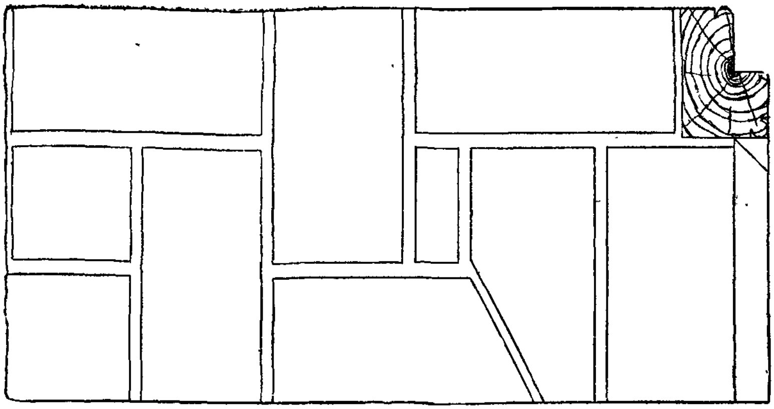


Fig. 264.—Enlarged Plan of Brick Bonding.

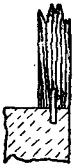


Fig. 265.—Stone Block for Door Post.

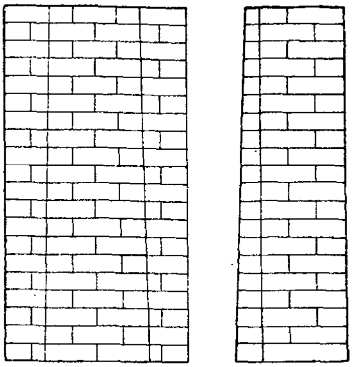


Fig. 266. Fig. 267.

Figs. 266 and 267.—Gauged Pilaster.

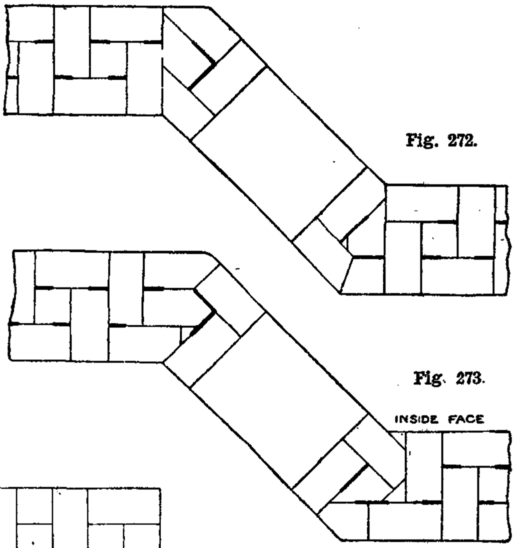


Fig. 272.

Fig. 273.

Figs. 272 and 273.—Bonding for Bay with opening in splayed side.

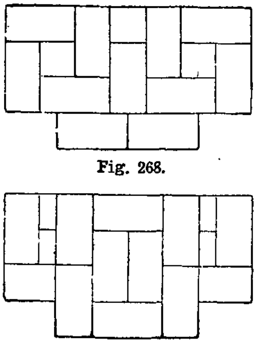


Fig. 268.

Fig. 269.

Figs. 268 and 269.—Bottom Courses of Gauged Pilaster.

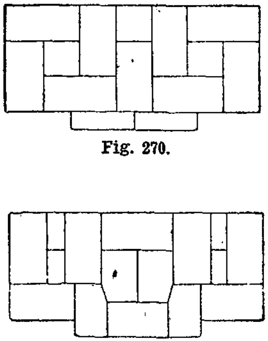


Fig. 270.

Fig. 271.

Figs. 270 and 271.—Top Courses of Gauged Pilaster.

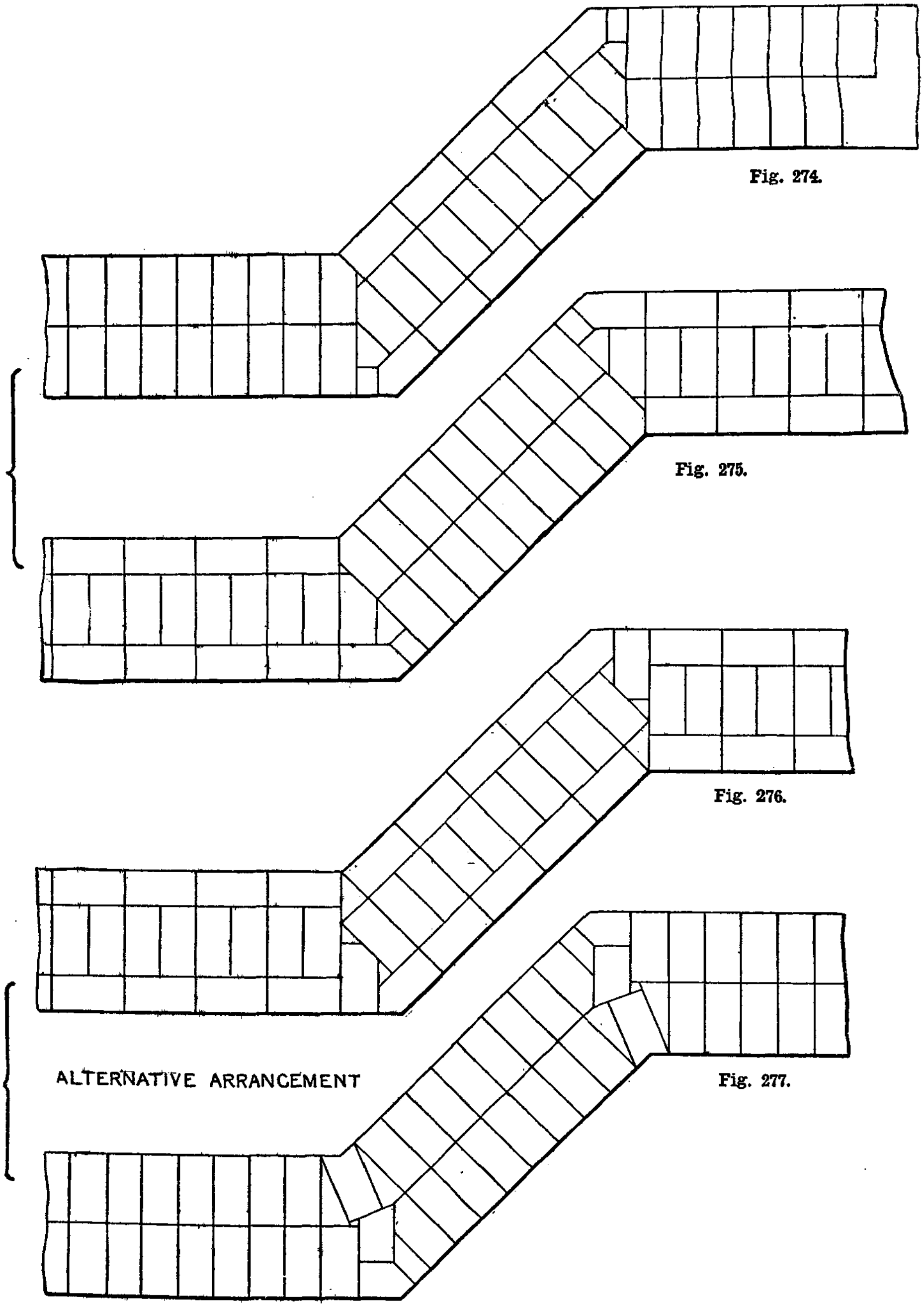

ALTERNATIVE ARRANGEMENT

Fig. 274.

Fig. 275.

Fig. 276.

Fig. 277.

Figs. 274 to 277.—Part of Brick Bay forming Three Sides of an Octagon.

posts, with a projecting cleat or stump let into the stone threshold or base. Stone blocks might also be used resting on the threshold and bonded in the brickwork to raise the feet of the door frame out of the reach of moisture, but this is not usual with ordinary doors, especially when arranged with a sill on the threshold, or for "labourers' cottages and artisans' dwellings." If stone blocks are used, they will be as shown in Figs. 263 and 265. The door frame would be temporarily stayed by nailing on strips of wood diagonally and transversely as shown, and by raking struts in the opposite direction, according to circumstances.

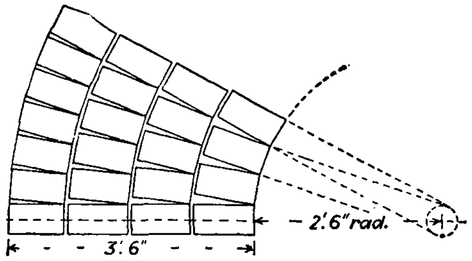


Fig. 278.

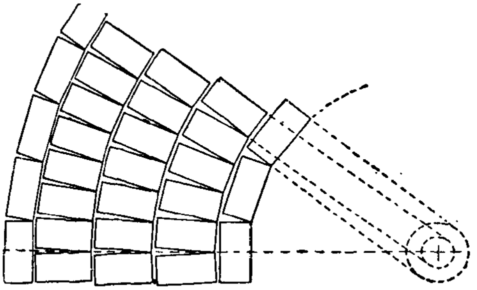


Fig. 279.

Figs. 278 and 279.—Courses of Brick Wall for Bridge Foundation.

Gauged Pilaster.

Front and side elevations of a gauged pilaster are presented by Figs. 266 and 267, and two adjacent courses, at the bottom and top respectively, are shown in Figs. 268 and 269 and Figs. 270 and 271. Both headers and stretchers may be reduced in width in the pilaster as the courses rise, or the stretchers only may be reduced.

Bonding Brickwork of Bay Window.

Figs. 272 and 273 show bonding that can be adopted for an octagonal bay not having a recess for the window-frame and not having a window in front of the bay, the opening to be covered by a camber arch. The thick lines show straight joints in the bonding. The camber arch over the opening would be an

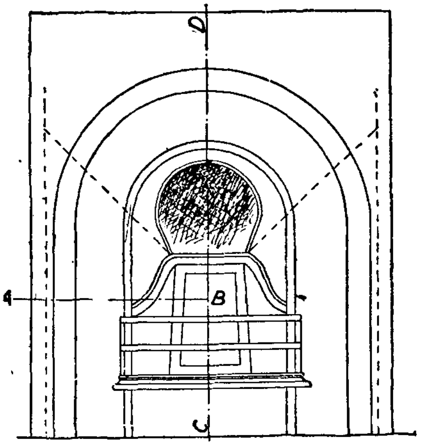


Fig. 280.

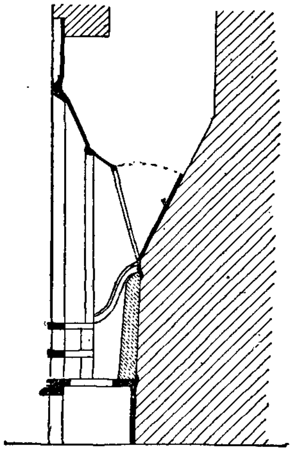


Fig. 281.

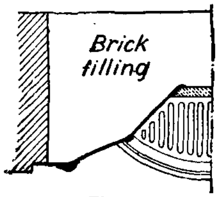


Fig. 282.

Figs. 280 to 282.—Front Elevation, Vertical Section, and Half Horizontal Section of Fireplace for Register Grate.

ordinary one, 12 in. deep, the joints being alternately 4 in. and 8 in. from the top. Camber arches should be used with discretion in projecting bays, as the thrust produced may force out the piers and cause the arches to drop. Alternative arrangements for the courses of a bay of solid 18-in. brickwork, forming three sides of an octagon and projected from an 18-in. wall, are illustrated by Figs. 274 to 277.

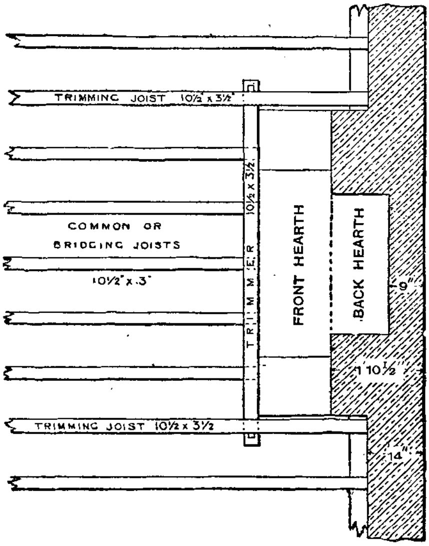

TRIMMING JOIST 10½ x 3½
COMMON OR BRIDGING JOISTS
10½" x 3"
TRIMMER 10½ x 3½
FRONT HEARTH
BACK HEARTH
9"
1' 10½"
14"
TRIMMING JOIST 10½ x 3½

Fig. 283.—Plan of Fireplace Opening.

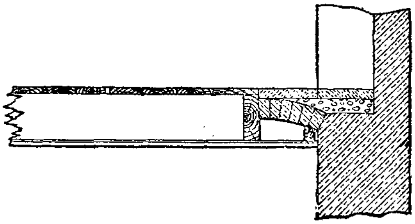


Fig. 284.—Section through Trimmer Arch.

Bonding Brick Well for Bridge Foundation.

The correct bond for the wall of a well for the foundation of a bridge is quite a matter of opinion. With a diameter of 12 ft. outside and 5 ft. inside, there would be a thickness of 3 ft. 6 in. each side, and the bricks being 10 in. by 5 in. by 3 in., it would appear that English bond, as shown in Figs. 278 and 279, would be suitable if well flushed up and grouted. The bonding for the corbel part should be similar.

Fireplace for Register Grate.

Fig. 280 shows an elevation, Fig. 281 a vertical cross section at the centre, and Fig. 282 a half horizontal cross section (just over the grate) of a good register grate fireplace. The firebrick back, if it gets broken, is picked out by inserting a knife or chisel behind the top, and the new one is simply pushed into place.

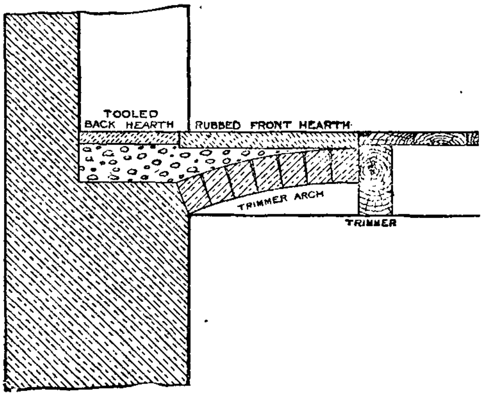

TOOLED BACK HEARTH
RUBBED FRONT HEARTH
TRIMMER ARCH
TRIMMER

Fig. 285.—Fireplace on Upper Floor.

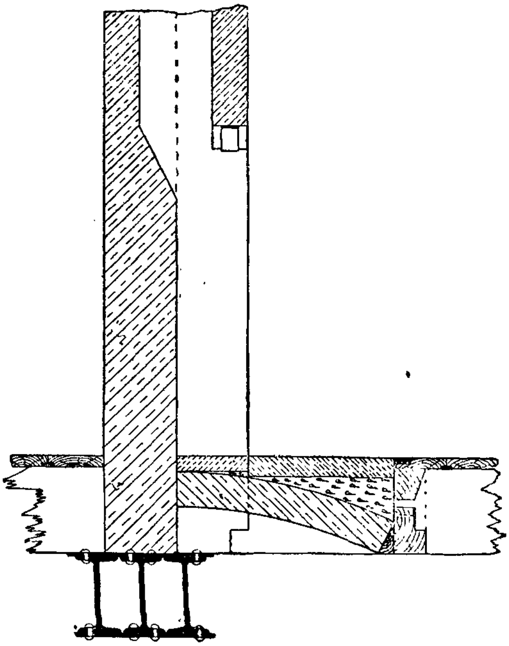


Fig. 286.—Fireplace in Wall over Girders.

The back of the grate is often left vacant, allowing soot to collect; it should be filled up as shown by dotted lines in Fig. 280. The register door is either pushed back or taken off when the chimney is to be cleaned.

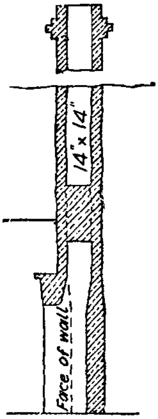

14" x 14"
Face of wall

Fig. 287.

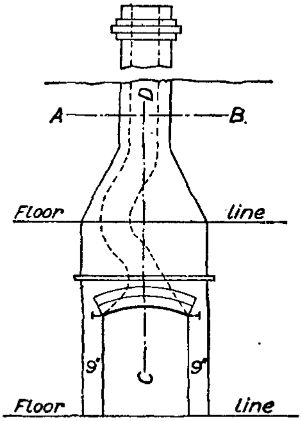
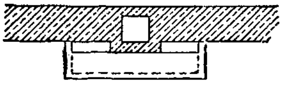

A
B
Floor
line
9"
9"
Floor
line

Fig. 288.



SECTION A—B
Fig. 289.

Figs. 287 to 289.—Chimney Flue for Cottage.

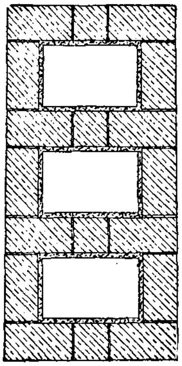


Fig. 290.

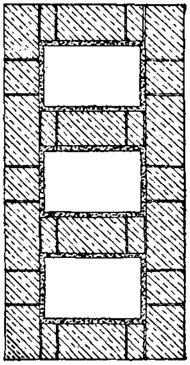


Fig. 291.

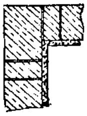


Fig. 292.

Figs. 290 to 292.—Bonding of Chimney Flues.

Supporting Front Hearth.

The method of carrying the front hearth of a first floor fireplace is shown by Figs. 283 and 284, the joists being trimmed round to a chimney breast 6 ft. 6 in. wide. They are supposed to run at right angles to the chimney breast. The room is assumed to be 18 ft. wide, and the construction is hidden by the ceiling below.

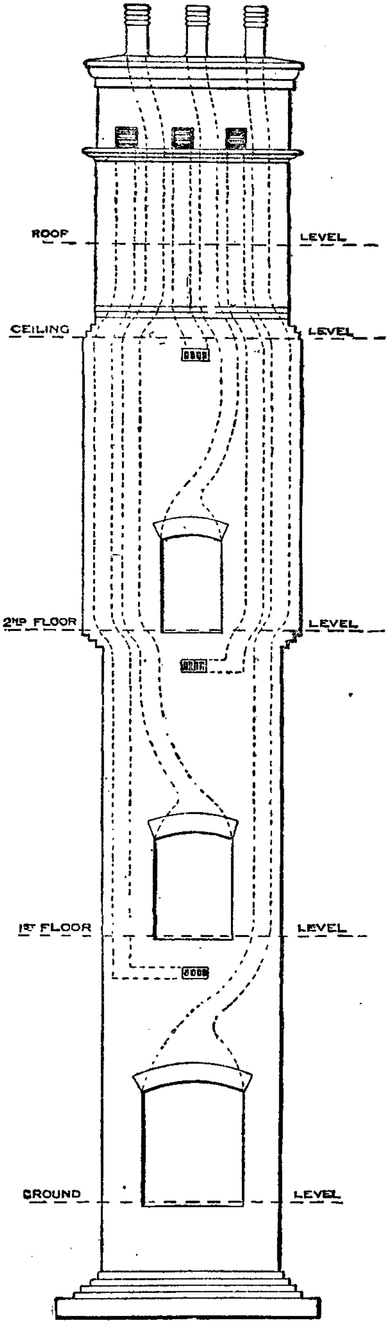

ROOF
LEVEL
CEILING
LEVEL
2ND FLOOR
LEVEL
1ST FLOOR
LEVEL
GROUND
LEVEL

Fig. 293.—Chimney Breast, Stack and Foul-air Flue.

Fireplace on Upper Floor.

Fig. 285 represents, to a scale of ½ in. to a foot, a vertical cross section through a fire-opening on an upper floor, showing depth of opening 14 in., back 9 in., a brick trimmer arch, and a 9-in. by 4-in. trimmer, also front and back hearths. A fireplace opening in a wall supported by girders is illustrated by Fig. 286.

Flues.

Flues for kitchens should be 14 in. by 9 in.; for other domestic fireplaces 9 in. by 9 in. is sufficient. Flues should be pargeted—rendered inside with parge or plaster to present a smooth surface and prevent direct smoke passing through external joints. Foul air or ventilation flues should be placed between smoke flues to utilise the warmth for an upward current. The division walls are called "withs" or "wythes."

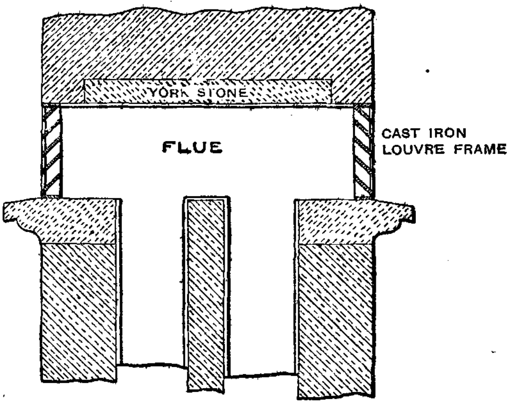


Fig. 294.—Enlarged Section of Foul-air Outlets.

Chimney Flue for Cottage.

A fireplace opening, 3 ft. 4 in. wide, is large enough for a kitchener or close range in a cottage; for an open grate that is to burn coals 2 ft. 6 in. should be sufficient; while 9 in. by 9 in. ought to be large enough for a short flue and an ordinary coal fire. If peat is used, a fireplace and flue of the sizes and shapes shown in Figs. 287 to 289 may be necessary.

Three Flues in Half-brick Chimney Stack.

Plans of two successive courses of a half-brick chimney stack of three flues (scale $\frac{1}{12}$) are shown by Figs. 290 and 291. The pargetting is clearly shown. An alternative is suggested in Fig. 292.

Chimney Breast, Stack, and Foul-air Flue.

Fig. 295 shows, to a scale of 8 ft. to an inch, an elevation of a chimney breast and stack above, rising through three storeys. A foul-air flue is carried up from each room. The

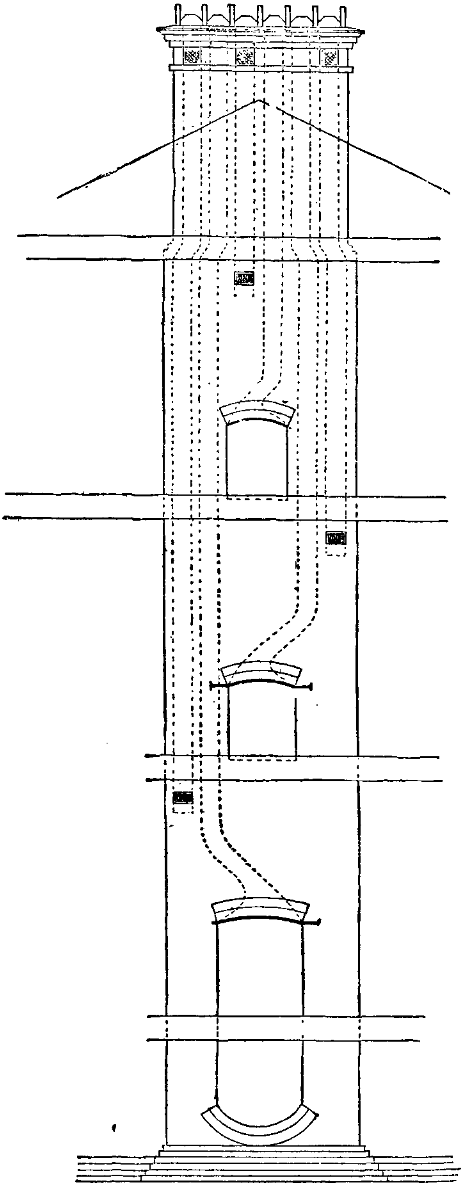


Fig. 295.—Another Design of Chimney Breast, Stack and Foul-air Flue.

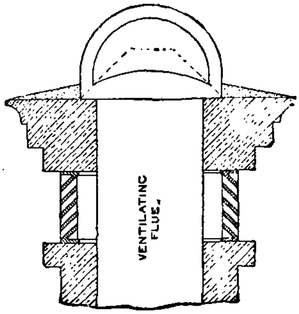

VENTILATING FLUE.

Fig. 296.—Section at Top of Ventilating Flue showing Elevation of Terminal Partition.

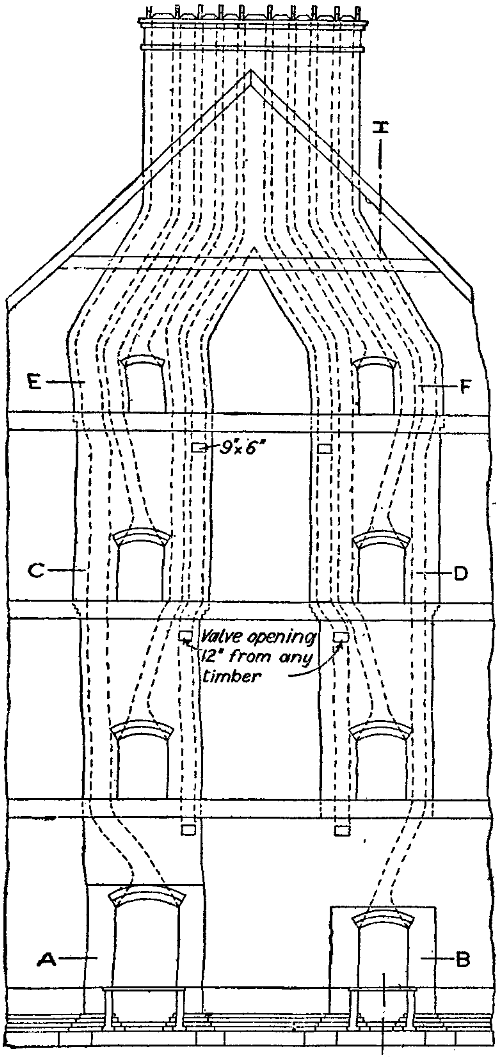

I
E
F
9"x6"
C
D
Valve opening 12" from any timber
A
B
G

Fig. 299.

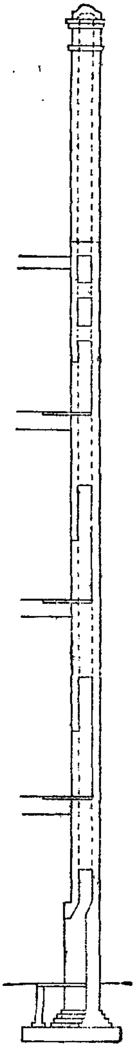


Fig. 300.

Fig. 297.—Outside Elevation of Mica Flap Inlet.

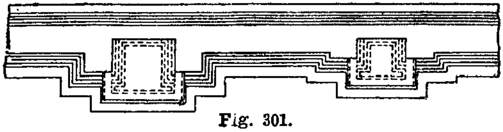


Fig. 301.

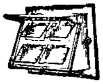


Fig. 298.—Inside View of Mica Flap Inlet.

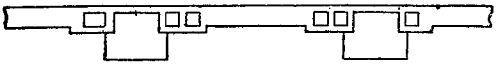


Fig. 302.

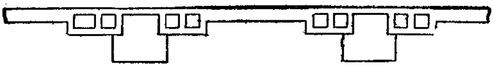


Fig. 303.

Figs. 299 to 303.—Elevation and Sections of Chimney Flues and Fireplaces for Eight-room House.

smoke and air flues are shown dotted in. A detail of the foul-air outlets is given by Fig. 294. The inlets to the ventilating flue should be Boyle's mica-flap inlets. Fig. 293 may be compared with Fig. 295, which shows a similar but slightly different arrangement. The inverted arch at the base will be noted. The chimney breast and shaft rise through the kitchen, and, together with a foul-air flue, through the sitting-room and bedroom, all the

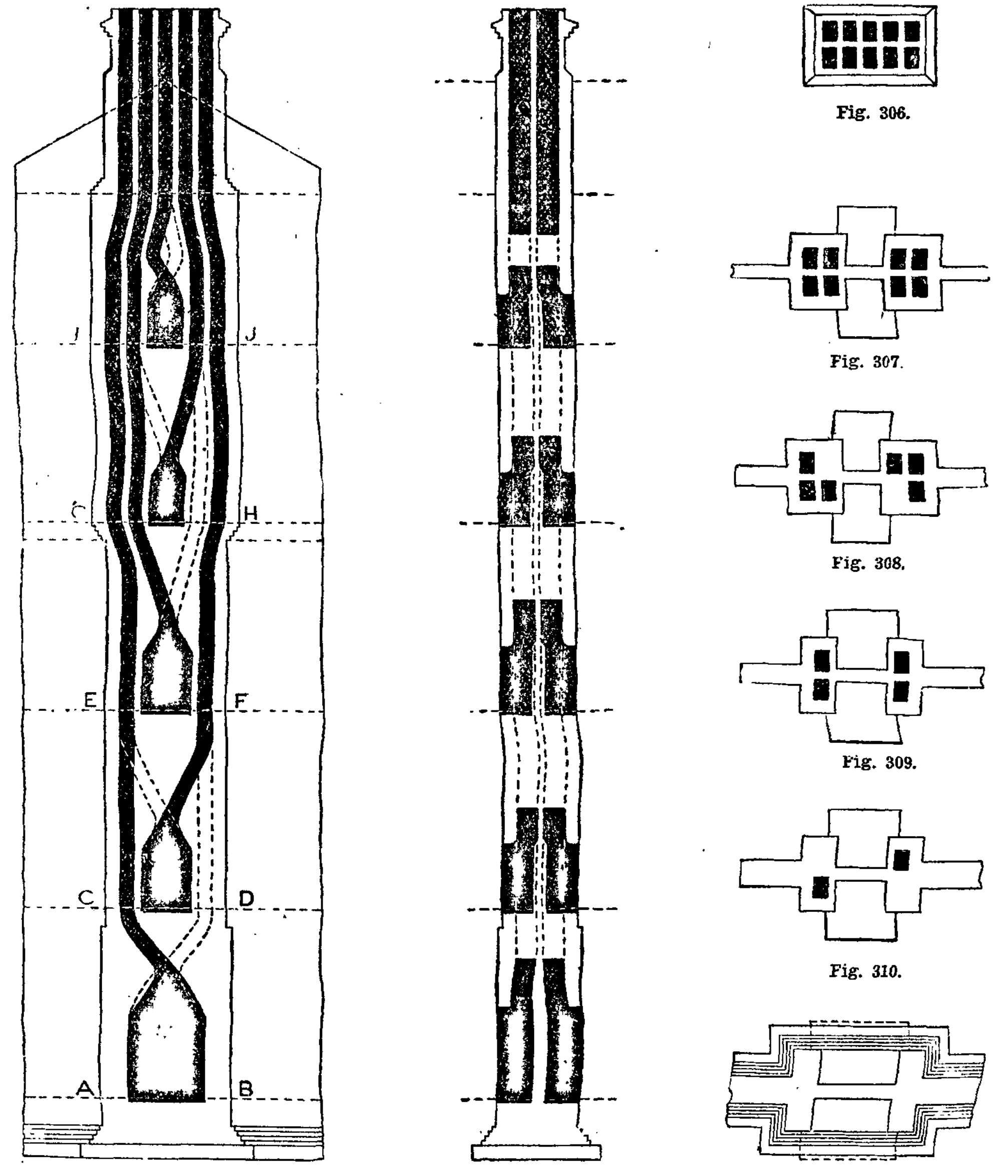


Fig. 304. Fig. 305. Fig. 306. Fig. 307. Fig. 308. Fig. 309. Fig. 310. Fig. 311.

Figs. 304 to 311.—Sections of Chimney Flues and Fireplaces for Two Houses, back to back.

rooms being 10 ft. high. Fig. 296 represents a section at the top of the ventilating flue, and shows an elevation of the terminal partition. An outside elevation and an inside view of one of the mica-flap inlets shown in position in Fig. 295 are presented by Figs. 297 and 298. Further explanatory figures showing flues and fireplaces are given on pp. 89 to 92; the inscriptions beneath them (see Figs. 299 to 317) are sufficient description.

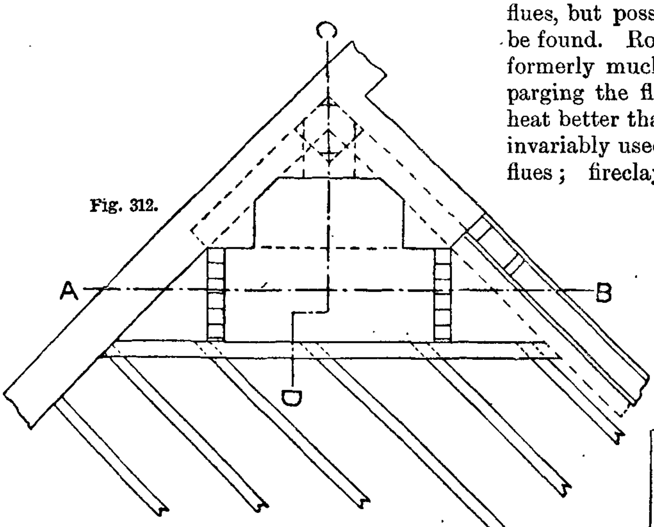


Fig. 312.

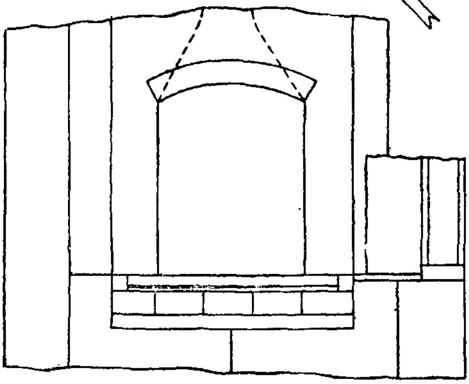


Fig. 313.

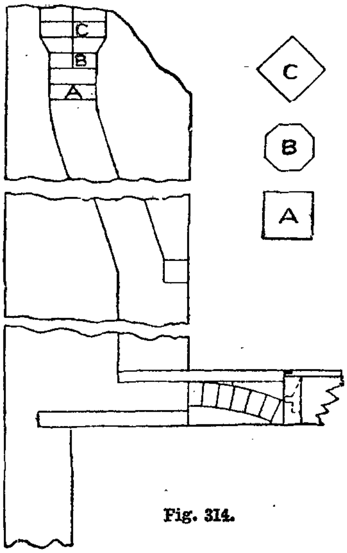


Fig. 314.

Figs. 312 to 314.—Sections of Fireplace and Flue in Angle of Building.

Flues in Hollow Brick Wall.

A hollow wall consisting of a 4½-in. skin outside a 9-in. wall may have a flue constructed in it as shown in Fig. 318. The system shown in Fig. 319 infringes most bye-laws, there being only one 4½-in. thickness at back of fireplace. However, where such a construction is allowed, the flue will be as shown.

Pargetting for Lining Flues, etc.

Cow-dung mortar is a preparation that has been universally approved for lining ordinary flues, but possibly some better substance may be found. Roman cement (or pozzuolana) was formerly much used for setting coppers and parging the flues, and was supposed to stand heat better than Portland cement. Fireclay is invariably used for setting furnaces and boiler flues; fireclay is friable, but not to any serious extent. For lining Bessemer converters, reverberatory furnaces, etc., where extreme heat is generated, a fettling of materials, such as ganister and broken firebrick, is used. More often the question is, which is the cheapest rather than which is the best material for lining ordinary flues; and although some of the fireproof plasters might be found to be more efficient than the ordinary pargetting, they could only be adopted by being expressly specified. By fireproof or fire-resisting materials are meant those materials that are refractory or incombustible. The materials that are used for preventing the passage of heat are called non-conducting materials, and many of them are highly combustible, such as hair-felt.

Coring a Chimney.

Coring a chimney consists in clearing it of any projections left by the pargetting. "Notes

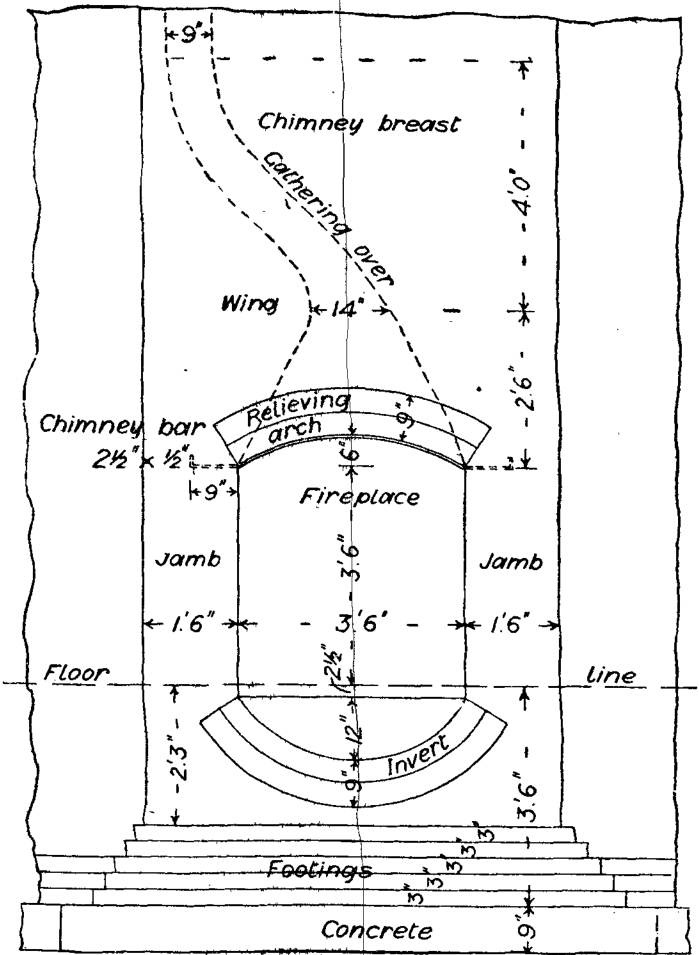

9"
Chimney breast
Gathering over
4'0"
Wing
14"
2'6"
Relieving arch
9"
Chimney bar
2½" × ½"
6"
9"
Fireplace
3'6"
Jamb
Jamb
1'6"
3'6'
1'6"
Floor line
2½"
2'3"
12"
Invert
9"
3'6"
3
3
3
3
Footings
Concrete
9"

Fig. 315.

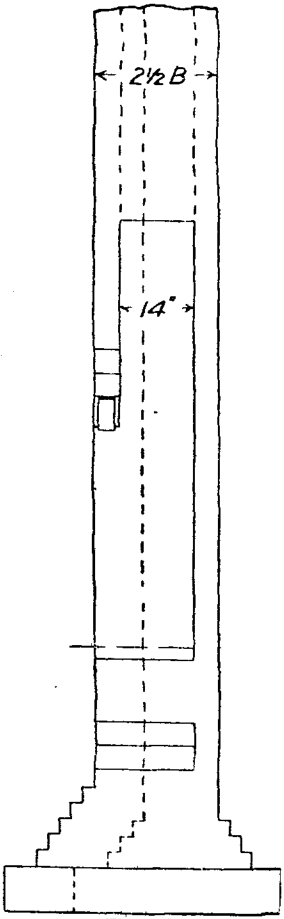


Fig. 316.

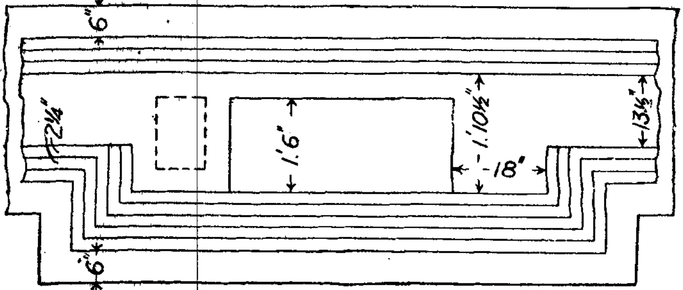

6"
2¼"
1'6"
1'10½"
18'
13½"
6"

Fig. 317.

Figs. 315 to 317.—Elevation, Section and Plan of Fireplace with Relieving and Inverted Arches.

on Building Construction" says:—"While a chimney flue is being built, it is advisable to keep within the chimney a bundle of rags or shavings, called a sweep, in order to prevent mortar from falling upon the sides of the chimney, and after the flue is finished a wire

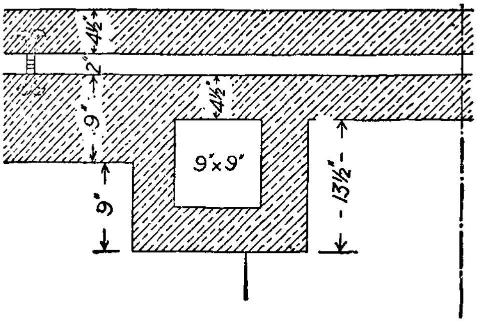

4½"
2"
9"
9"
4½"
9"x9"
13½"

Fig. 318.—Flue in Hollow Wall.

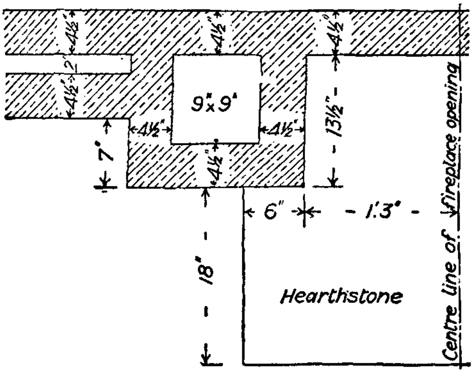

4½"
2"
4½"
4½"
4½"
9"x9"
4½"
4½"
4½"
13½"
7"
6"
1'3"
18"
Hearthstone
Centre line of fireplace opening

Fig. 319.—Flue in improperly constructed Hollow Wall.

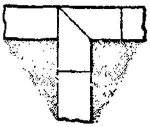


Fig. 320.

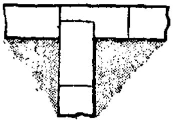


Fig. 321.

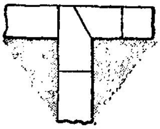


Fig. 322.

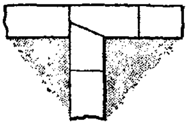


Fig. 323.

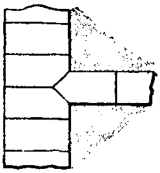


Fig. 324.

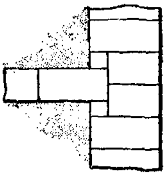


Fig. 325.

Figs. 320 to 325.—Bonds for Chimney Flues.

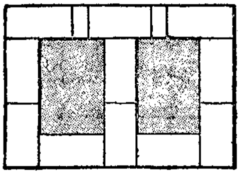


Fig. 326.

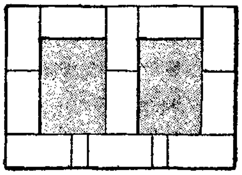


Fig. 327.

Figs. 326 and 327.—Two Flues in Chimney Bond.

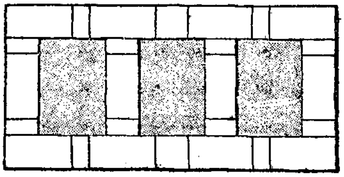


Fig. 328.

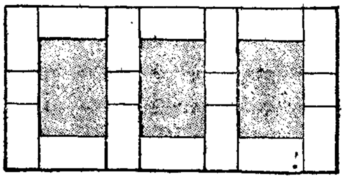


Fig. 329.

Figs. 328 and 329.—Three Flues in Flemish Bond.

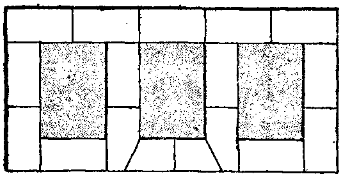


Fig. 330.

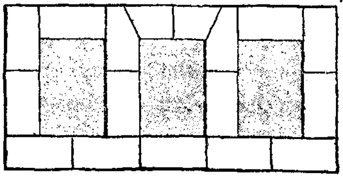


Fig. 331.

Figs. 330 and 331.—Three Flues in Chimney Bond.

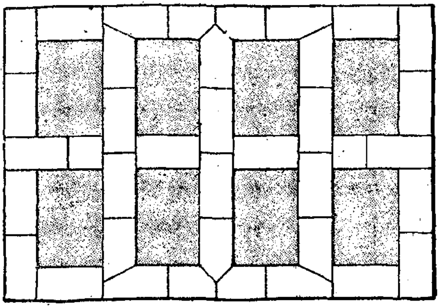


Fig. 332.

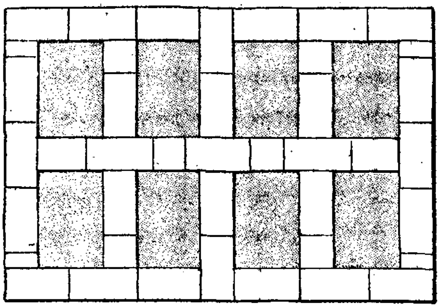


Fig. 333.

Figs. 332 and 333.—Eight Flues in Chimney Bond.

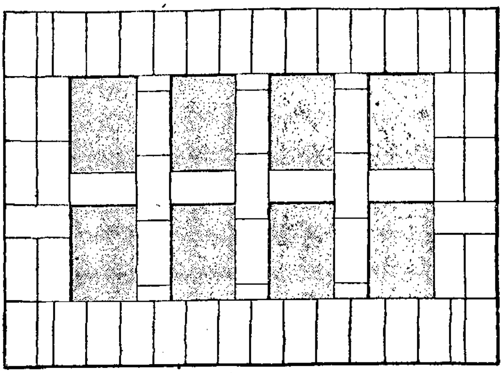


Fig. 334.

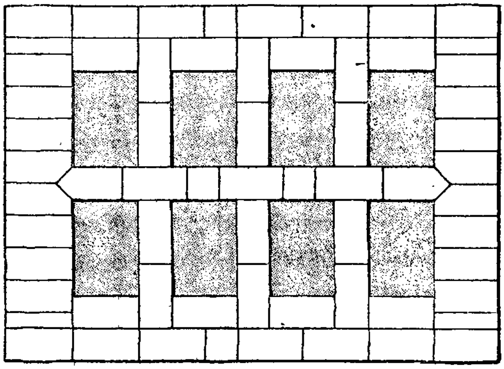


Fig. 335.

Figs. 334 and 335.—Eight Flues in English Bond with 9-in. Outer Wall.

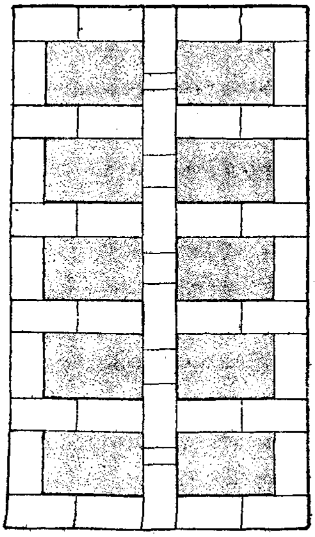


Fig. 336.

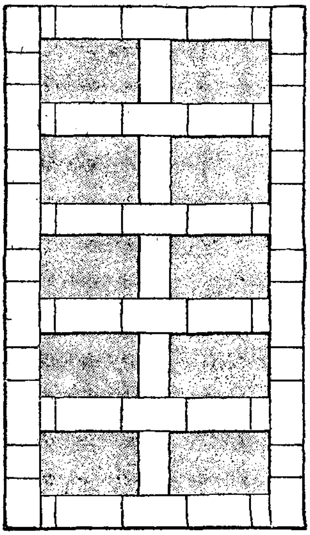


Fig. 337.

Figs. 336 and 337.—Ten Flues in Flemish Bond.

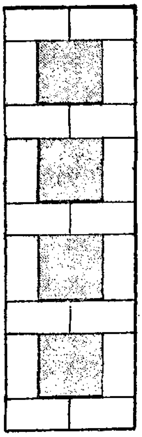


Fig. 338.

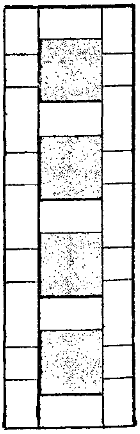


Fig. 339.

Figs. 338 and 339.—Four Flues in Flemish Bond.

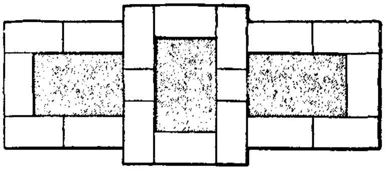


Fig. 340.

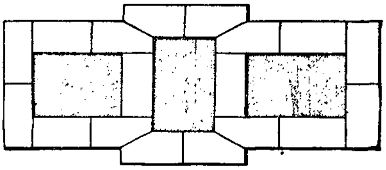


Fig. 341.

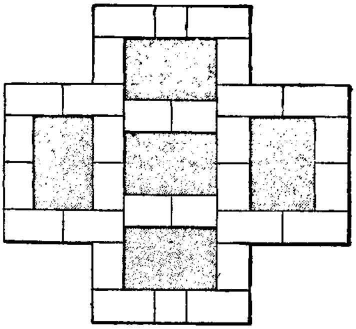


Fig. 342.

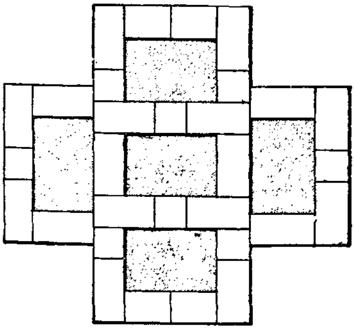


Fig. 343.

Figs. 340 to 347.—Ornamental Chimney Stacks and Grouped Flues; Alternate Courses.

brush or core should be passed through the flue to clear away small irregularities and to discover any obstruction that may exist in the flue."

Grouped Flues and Stacks, Plain and Ornamental.

Typical bonds for flues are illustrated by Figs. 320 to 325; and successive courses of

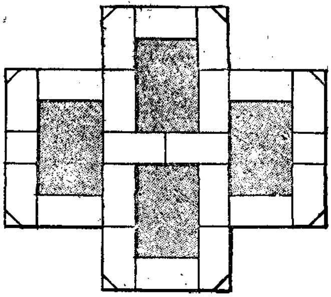


Fig. 344.

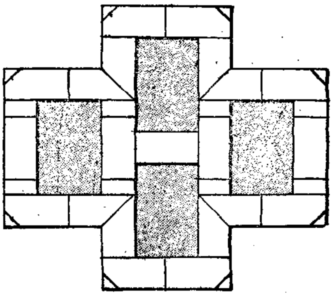


Fig. 345.

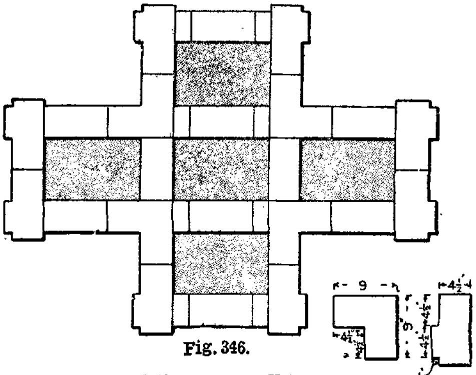


Fig. 346.

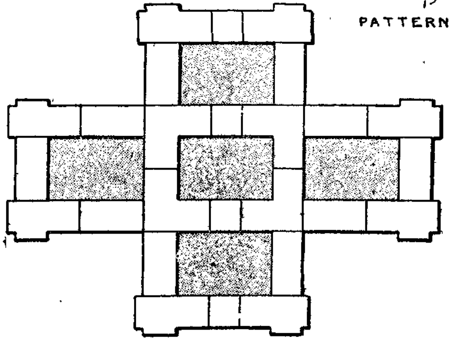


Fig. 347.

plain grouped flues are also shown as follow: Fig. 326 and 327, two 14-in. by 9-in. flues in chimney bond; Fig. 328 and 329, three 14-in. by 9-in. flues in Flemish bond; Figs. 330 and 331, three 14-in. by 9-in. flues in chimney bond; Figs. 332 and 333, eight flues in chimney bond; Figs. 334 and 335, eight flues in English bond with a 9-in. outer wall; Figs. 336 and 337, ten flues in Flemish bond, and Figs. 338 and 339, four flues in Flemish bond. Easily understood groupings of flues largely built with ornamental bricks are represented by Figs. 340 to 355; these figures are arranged to show successive courses of the different arrangements and make the bonding quite clear.

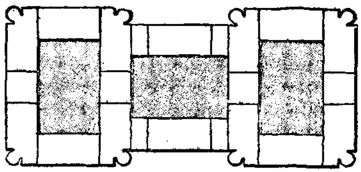


Fig. 348.

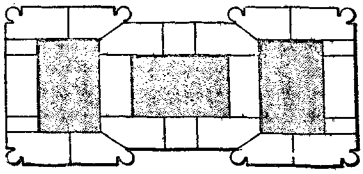


Fig. 349.

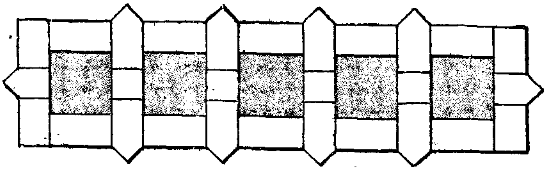


Fig. 350.

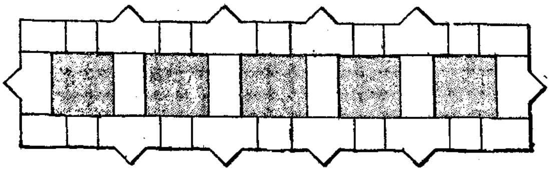


Fig. 351.

Ornamental Chimney Stack.

Fig. 356 shows the elevation and sectional plan of an ornamental brick chimney stack containing three flues. For the bondings see Figs. 357 and 358. The stack is built up at the gable of a house.

Weathering Bonds for Brick Chimneys.

Three elevations for the bonding of the weathered portion of a chimney are presented by Figs. 359 to 361, and of these illustrations, perhaps Fig. 361 is to be preferred. By having

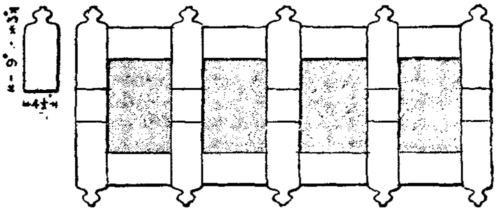


Fig. 352.

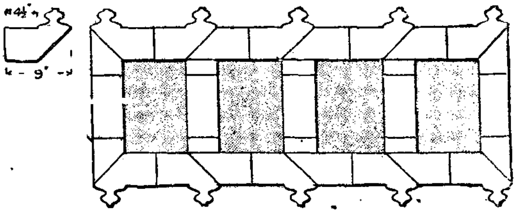


Fig. 353.

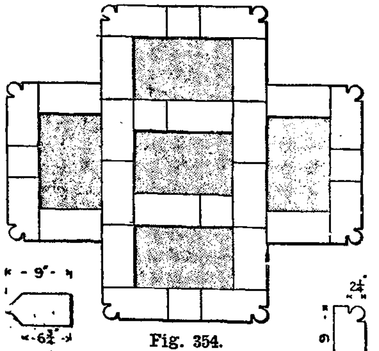


Fig. 354.

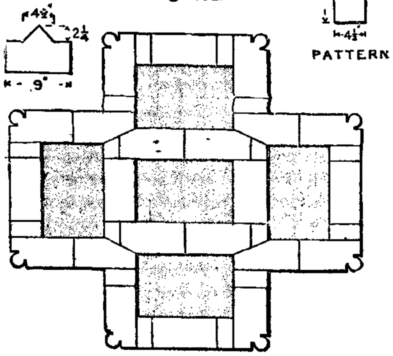
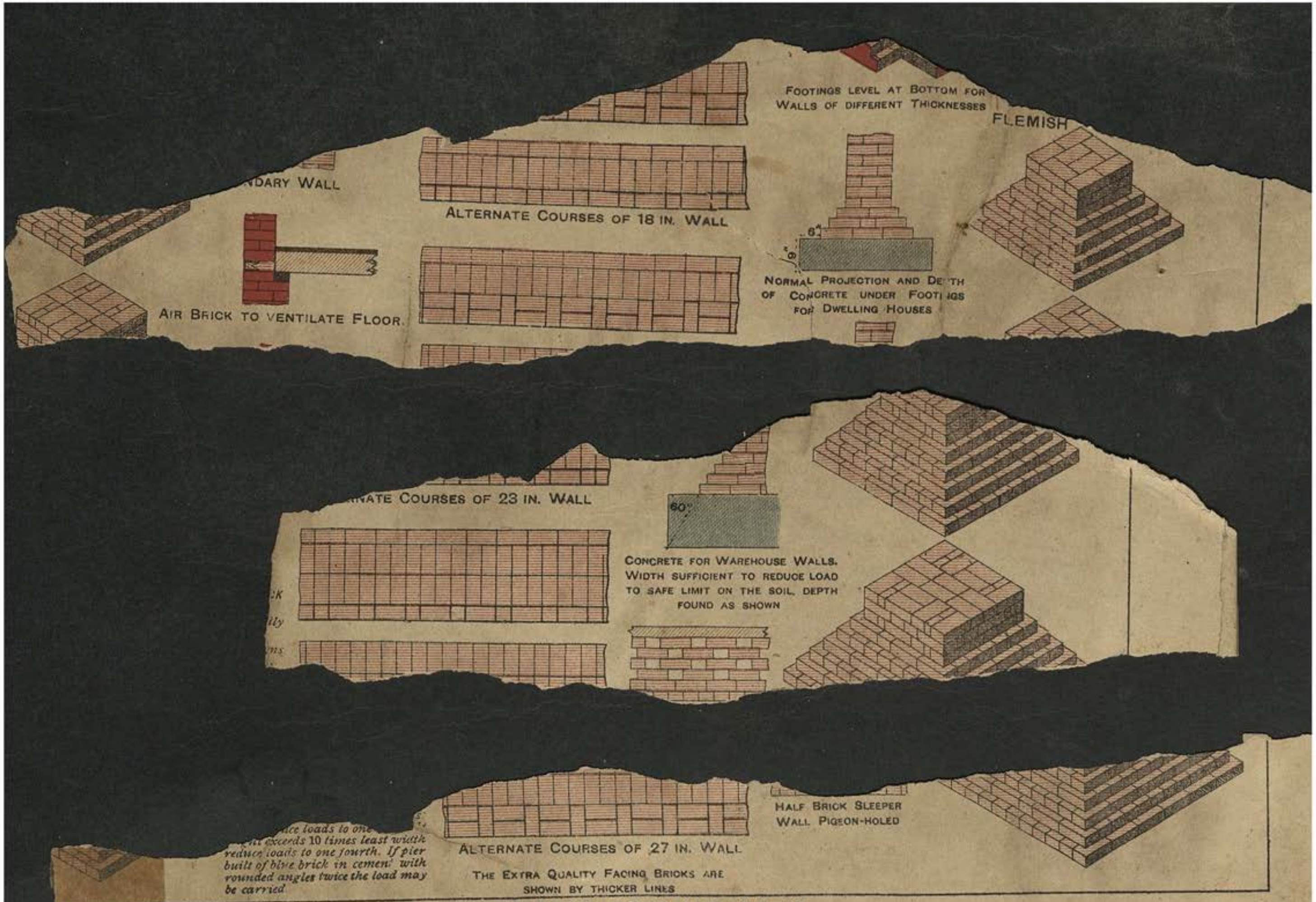


Fig. 355.

Figs. 348 to 355.—Ornamental Chimney Stacks and Grouped Flues; Alternate Courses.


FOOTINGS LEVEL AT BOTTOM FOR WALLS OF DIFFERENT THICKNESSES
FLEMISH
NDARY WALL
ALTERNATE COURSES OF 18 IN. WALL
6"
9"
NORMAL PROJECTION AND DE TH OF CONCRETE UNDER FOOTI GS FOR DWELLING HOUSES
AIR BRICK TO VENTILATE FLOOR.
NATE COURSES OF 23 IN. WALL
60"
CONCRETE FOR WAREHOUSE WALLS. WIDTH SUFFICIENT TO REDUCE LOAD TO SAFE LIMIT ON THE SOIL. DEPTH FOUND AS SHOWN
CK
lly
ns
HALF BRICK SLEEPER WALL PIGEON-HOLED
ce loads to one
exceeds 10 times least width
reduce loads to one fourth. If pier
built of blue brick in cement with
rounded angles twice the load may
be carried
ALTERNATE COURSES OF 27 IN. WALL
THE EXTRA QUALITY FACING BRICKS ARE SHOWN BY THICKER LINES

BONDING OF BRICK PIERS AND WALLS.

the weathered part 2 ft. high, a better arrangement, giving three insets, can be made. The arrangement shown in Fig. 362 is the best when the weathered part is 2 ft. 3 in. high.

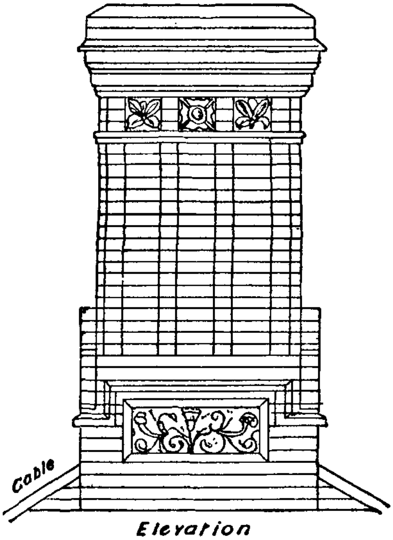


Fig. 356.—Ornamental Chimney Stack.

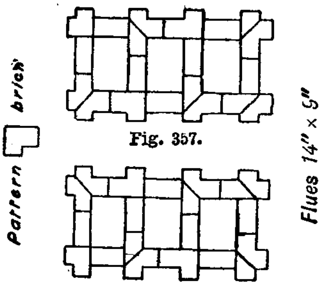


Fig. 357.

Fig. 358.

Figs. 357 and 358.—Courses of Ornamental Stack.

Brick Chimney Caps.

Three ugly designs for brick chimney caps are presented by Figs. 363 to 365 as solutions to an ill-considered examination question. The illustrations show end elevations of the top of the shaft down to two courses below the necking, which is three courses in depth—two plain and one weathered; the plain courses are bonded as in Figs. 366 and 367, the letters referring to Fig. 363. Above the necking are eight plain courses, then the cap, consisting of three sailing courses, four plain courses, and three splayed.

Moulded Bricks and Ornamental Brickwork.

Whilst in general retaining the dimensions of the hand-made and machine-pressed bricks

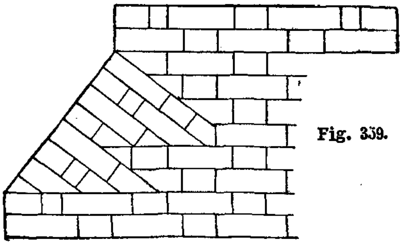


Fig. 359.

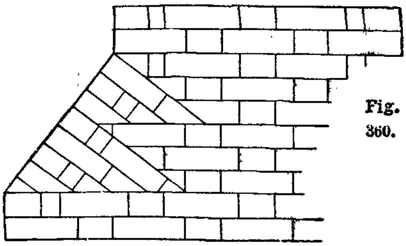


Fig. 360.

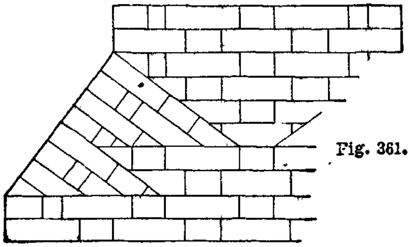


Fig. 361.

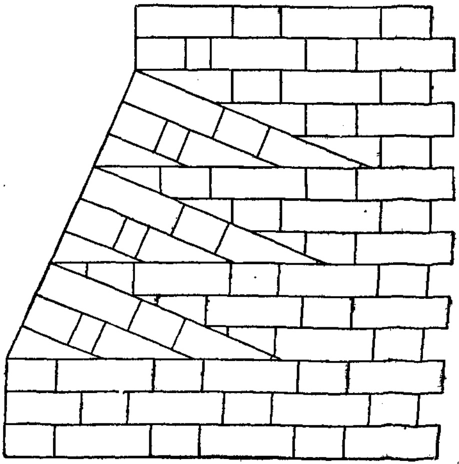


Fig. 362.

Figs. 359 to 362.—Alternative Methods of Weathering Brick Chimney.

(Figs. 368 and 369), moulded bricks are produced in a great variety of decorative forms, as instanced by Figs. 370 to 393, which show merely the kinds used for string courses, cornices, etc. In addition, there are perforated bricks (Figs. 394 and 395); dog-toothed bricks (Fig. 396); bull-nose bricks (Fig. 397); chamfer bricks (Fig. 398); plinth bricks (Fig. 399); quoin bricks (Fig. 400); hollow quoin bricks (Fig. 401); roll-edge bricks (Fig. 402); roll and fillet bricks (Fig. 403); and others. Special moulded bricks of the shapes shown in Figs. 404 to 408 are used for copings. Examples of cornices built up with some of the moulded bricks already mentioned are illustrated by Figs. 409 to 413. A moulded cornice by Candy and Co. is illustrated by Fig. 414, and a panel of three 9-in. squares, by the Rowlands Castle Brick and Tile Co., by Fig. 415. The top of an 80-ft. water tower to the design of the writer is shown by Figs. 416 and 417.

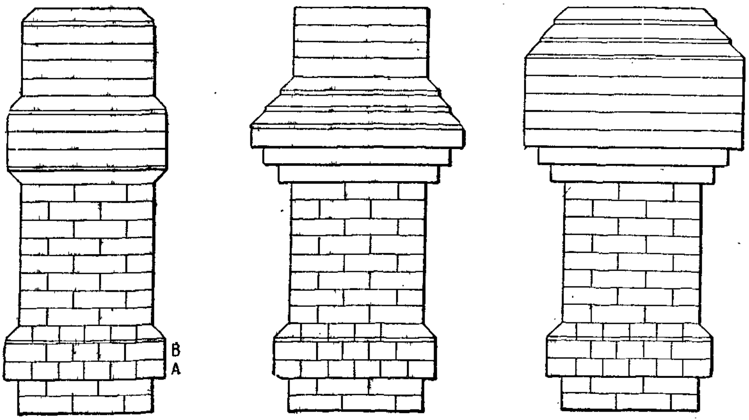


Fig. 363. Fig. 364. Fig. 365.

Figs. 363 to 365.—Ugly Designs for Chimney Cap.

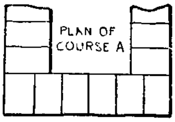


Fig. 366.

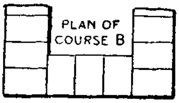


Fig. 367.

Figs. 366 and 367.—Courses of Chimney Cap.

Dog's-tooth Course and Indent-toothing.

Projecting points like teeth are given by a series of indents, such as a dog-tooth course under an oversailing course in brickwork, or a dentil course in a cornice. Fig. 418 illustrates a dog's-tooth course, and Fig. 419 a dentil course. A single raking course of dog's-tooth work is sometimes built up the sides of a gable under a brick-on-edge course. Fig. 420 shows a rather unusual case of dog's-tooth work on a gable wall. Toothing is the ordinary name for the bonding spaces left in the vertical end of a front wall when adjoining premises are to be built subsequently, or the holes cut in an existing wall for bonding a new wall to it. Indent-toothing strictly is the leaving of a series of spaces in a wall for the purpose of bonding another wall to it later on.

Determining Dimensions of Tall Chimney Shaft.

A chimney shaft is not so simple a structure that it can be satisfactorily designed for a given height without detailed knowledge of the duty that the chimney has to perform and the soil on which the chimney will stand; the size and number of the boilers, or the total furnace area, or the number of tons of coal that will be burnt per week, or some other measure of the work that has to be done, must also be known, together with the length of the flue, or the horizontal distance from the end of the boilers to the chimney shaft. Then the shape of the cross section has to be determined, whether square, octagonal or round; whether plain or

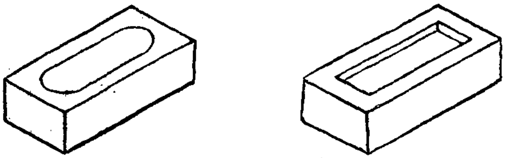


Fig. 368. Fig. 369.

Fig. 368.—Hand-made Brick.
Fig. 369.—Machine-pressed Brick.

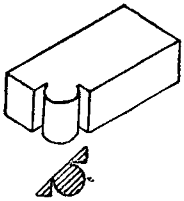


Fig. 370.

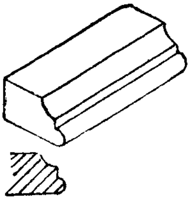


Fig. 371.

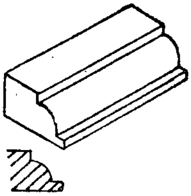


Fig. 372.

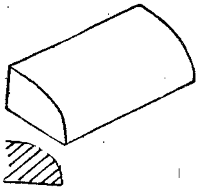


Fig. 373.

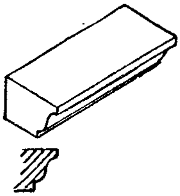


Fig. 374.

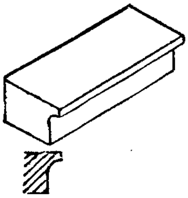


Fig. 375.

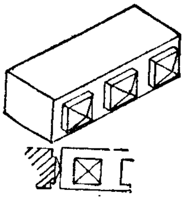


Fig. 376.

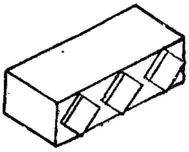


Fig. 377.

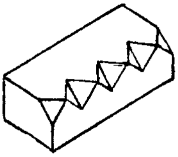
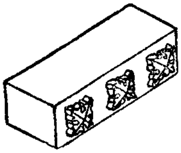


Fig. 378.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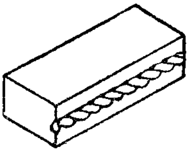


Fig. 380.

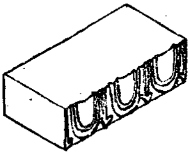


Fig. 381.

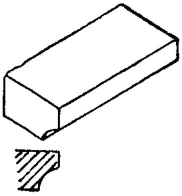


Fig. 382.

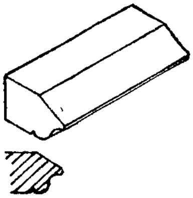


Fig. 383.

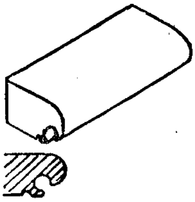


Fig. 384.

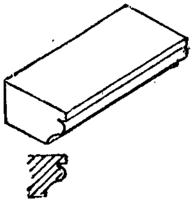


Fig. 385.

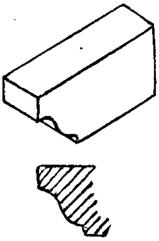


Fig. 386.

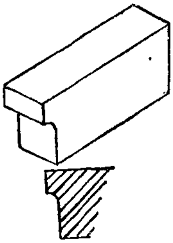


Fig. 387.

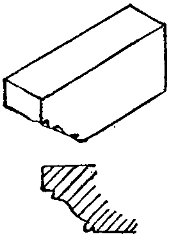
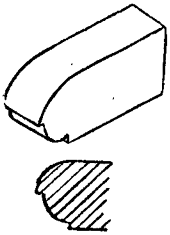


Fig. 388.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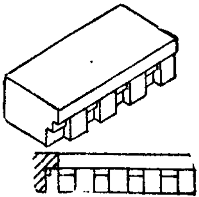
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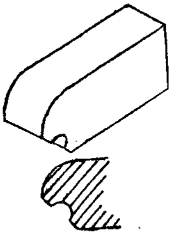


Fig. 391.

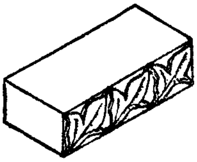


Fig. 392.

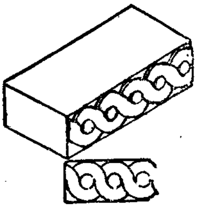


Fig. 393.

Figs. 370 to 393.—Moulded Bricks.

ornamental, with or without plinths, and the size and level of the flue, etc. In building a 60-ft. stack independent firebrick lining should be carried at least 20 ft. up the chimney.

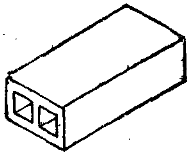


Fig. 394.

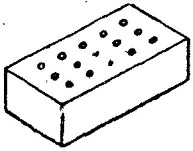
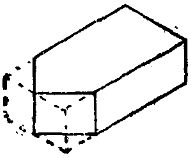


Fig. 395.

Figs. 394 and 395.—Perforated Bricks.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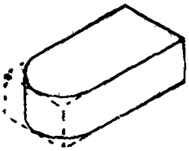
Fig. 397.

Fig. 396.—Dog-toothed Brick.
Fig. 397.—Bull-nose Brick.

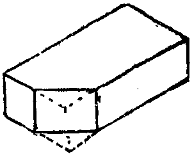
Fig. 398.

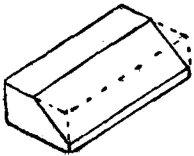
Fig. 399.

Fig. 398.—Chamfer Brick.
Fig. 399.—Plinth Brick.

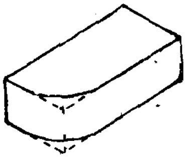
Fig. 400.

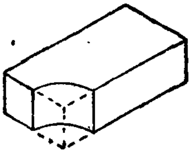
Fig. 401.

Fig. 400.—Quoin Brick.
Fig. 401.—Hollow Quoin Brick.

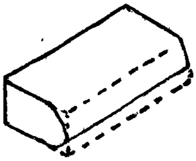
Fig. 402.

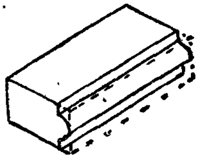
Fig. 403.

Fig. 402.—Roll-edge Brick.
Fig. 403.—Roll and Fillet Brick.

The net sectional area inside the chimney should be not less than one-tenth the gross furnace area of those boilers in use at one time. The outside batter should be not less than 2½ in. every 10 ft. of height. The width of the base should be not less than one-tenth of the height. The brickwork must be at least 9 in. thick for the top 20 ft., and set off 4½ in. thicker at every 20 ft. below.

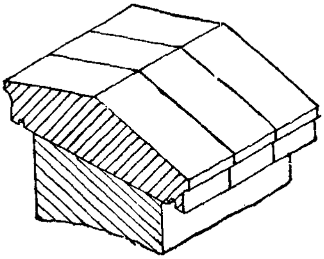
Fig. 404.—Moulded Coping Brick.

By the rules of the London County Council, every furnace chimney shaft "shall be at least 8½ in. in thickness at the top of the shaft and for not exceeding 20 ft. below, and shall be increased at least 4½ in. in thickness for every 20 ft additional height measured downwards."

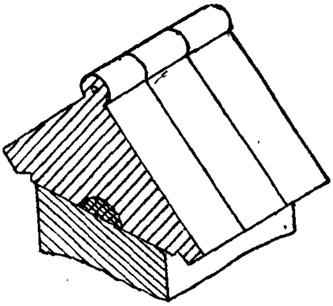
Fig. 405.—Moulded Coping Brick.

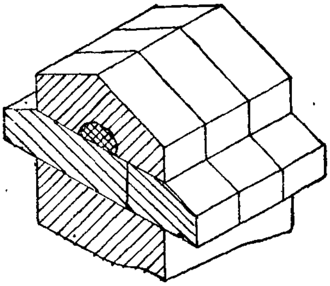
Fig. 406.—Moulded Coping Brick.

Foundation for Chimney Shaft.

The area covered by the foundation of a tall chimney shaft will depend chiefly on the nature of the substratum. The pressure upon the soil should not exceed from 1 to 5 tons per square foot, according to its character, the

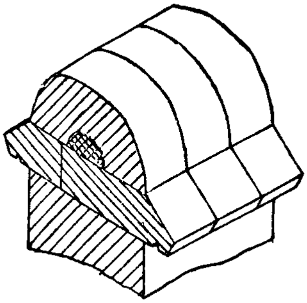
Fig. 407.

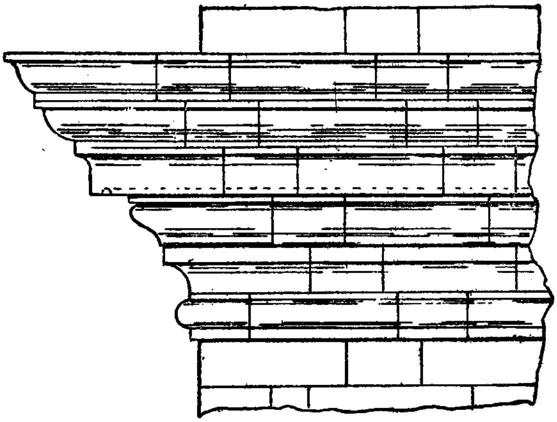
Fig. 409.

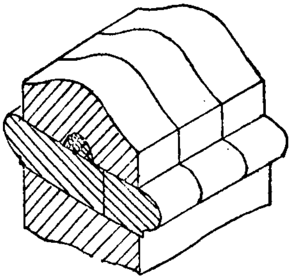
Fig. 408.

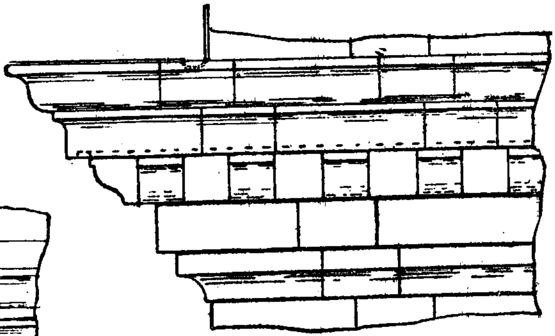
Fig. 410.

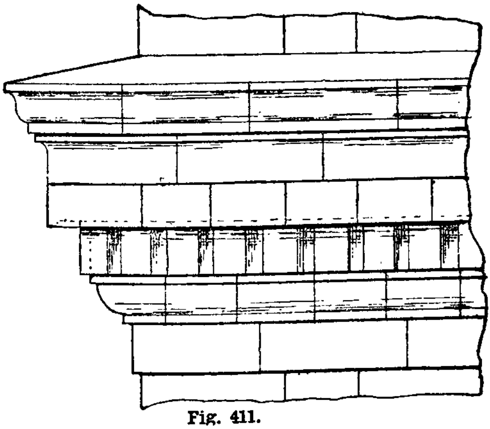
Fig. 411.

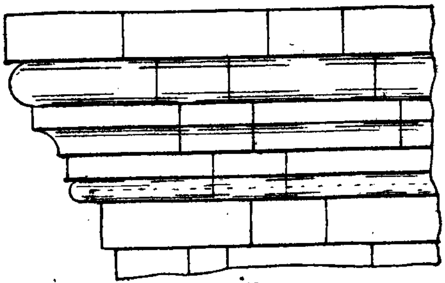
Fig. 413.

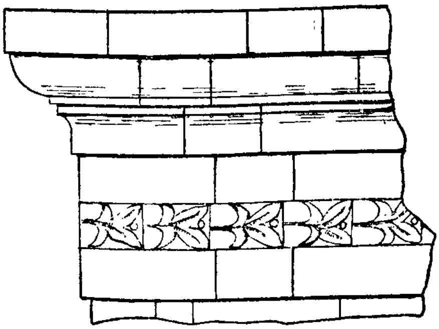
Fig. 412.

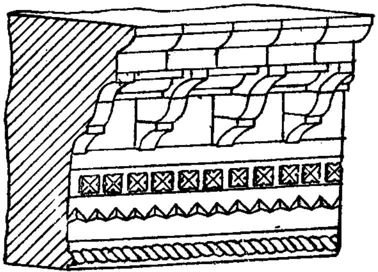
Fig. 414.

Figs. 407 and 408.—Moulded Coping Bricks. Figs. 409 to 413.—Brick Cornices.
Fig. 414.—Ornamental Brick Cornice.

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former upon a light sandy or loamy soil, and the latter upon a good gravel, deep firm clay, or rock. The thickness of concrete (one cement to six or seven ballast) should not be less than one and a half times its projection beyond the bottom course of footings, and the footings should be in not more than 2¼ in. set-offs. The concrete for a 100-ft. stack, 10 ft. wide at the base, might possibly be about 15 ft. square and 2 ft. 6 in. to 3 ft. thick.

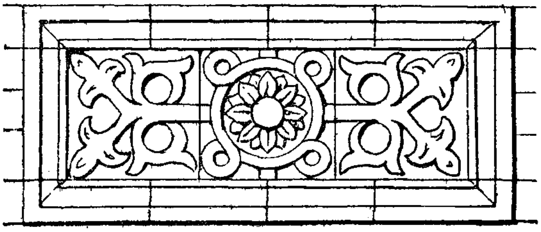


Fig. 415.—Moulded Panel.

Brickwork of Tall Chimney Shaft.

A chimney 90 ft. high from the ground level and 5 ft. 6 in. inside diameter must have 9 in. thickness of brickwork at the top, increased to 13½ in. at 20 ft. down, 18 in. at 40 ft. down, 22½ in. at 60 ft. down, 27 in. at 80 ft. down; the shaft should not be less than 1 in 48, so that the outside diameter at the ground line will be not less than 11 ft. The same widths and thicknesses may be adopted if the chimney is square in section. The cornice must not project more than 9 in. The foundation is the most important part of a chimney stack, and must be carefully considered. Now assume the case of a square chimney, 130 ft. high, which is to be built on the solid clay. The chimney is to measure 3 ft. square inside at the top. The top 20 ft. of the chimney will be a brick and a half thick, but if the chimney is only 120 ft. high the top length may be one brick thick. The thickness of the brickwork must be increased by a half brick every 20 ft. downwards. The chimney cap must not project more than the thickness of the chimney at the top. The width of the chimney outside at the ground level must be not less than one-tenth the height. The sides of the chimney must batter not less than 2½ in. in 10 ft. The firebrick lining must be independent of the outside skin, and not less than 1½ in. away from it, the space between the skin and the lining being protected from falling dirt by a sailing course over the top of the lining. Eight

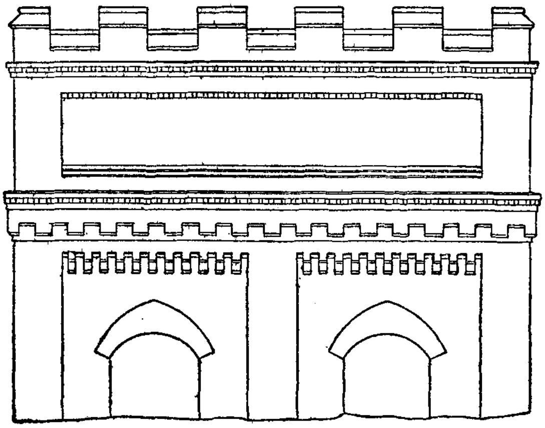


Fig. 416.

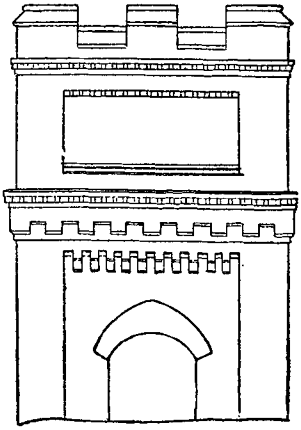


Fig. 417.

Figs. 416 and 417.—Top of Water Tower in Moulded Bricks.

that is, from the foundation to 10 ft. above ground level, the thickness exclusive of firebrick lining and air-space between must not be less than 27 in., and may be reduced 4½ in. as each 20 ft. upward is reached. The batter of courses of footings, concrete 20 ft. square and 6 ft. thick, top of concrete 10 ft. below ground level. Fig. 421 shows a section of the stack. A section of a badly designed 150-ft. stack is shown in Fig. 422.

Number of Bricks Required for Round Chimney Shaft.

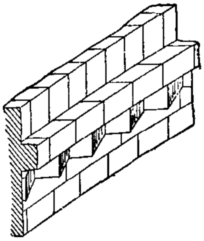


Fig. 418.—Dog-tooth Course.

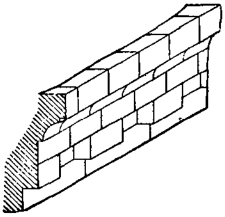


Fig. 419.—Dentil Course.

To find the quantity of bricks required, after the design and bonding are chosen, only simple rules are needed. Take the outside diameter in inches at the middle of each length, subtract the thickness, then multiply by the thickness and by 3·1416, and divide by 144 to bring it to square feet, and lastly multiply by the length of the section in feet to find the bulk in cubic feet. Do the same with each section, total up, and multiply by 14·2 for the total number of bricks. The proportion of headers and stretchers will depend upon the bond selected, and can be ascertained by taking a given length of one or two courses and counting up the number of each.

Bonding Tall Chimney Shafts.

There is no standard bond for tall chimney shafts; they are built in English, Flemish, or mixed bond, sometimes with an excess of stretchers, and at other times with an excess of headers, according to personal preference. One course of headers to four courses of stretchers makes a good and suitable bond for circular shafts. Where a difficulty is found in laying common headers, some architects prefer compass bricks, or bricks tapering in plan, for the headers, to facilitate the laying, and to produce close joints. Square shafts may be built in either English or Flemish bond without difficulty. The 450-ft. chimney at St. Rollox, Glasgow, is built in old English bond. The 300-ft. chimney at Johnson's Cement Works, Greenhithe, is built in Flemish bond. At Gostling's Cement Works, Northfleet, a

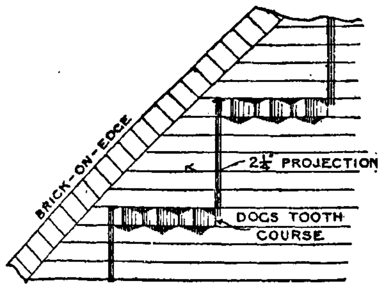


Fig. 420.—Dog-tooth Course, Brick-on-Edge Course, etc.

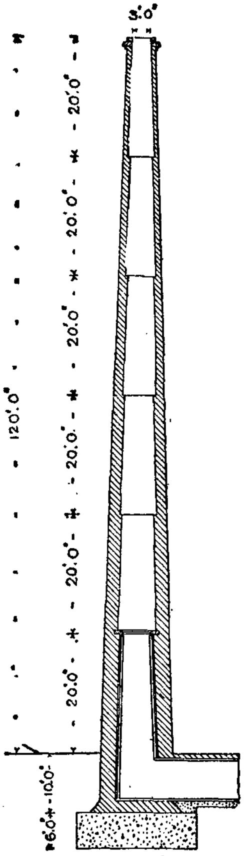


Fig. 421.

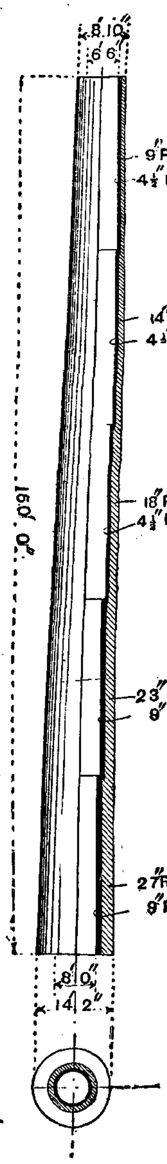


Fig. 422.

Figs. 421 and 422.—Sections of Tall Chimney Shafts.

220-ft. chimney is built in stretching bond. At Barker's Brick Works, Worcester, the 160-ft. chimney is built with three stretchers to one header. At the Surrey Commercial Docks the 110-ft. chimney is constructed with all headers on the circular face. For the 100-ft. chimney at Farringdon Street Goods Station Beart's patent perforated radiating bricks were used in the circular shaft, laid all headers on the external face.

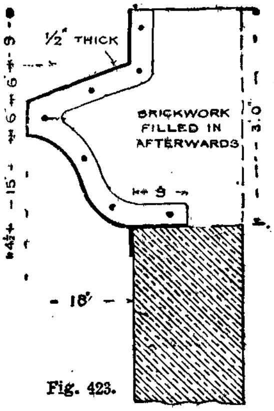


Fig. 423.

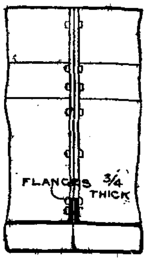


Fig. 424.

Figs. 423 and 424.—Cast-iron Capping for Chimney Shaft.

Cast-Iron Capping for Chimney Shaft.

Figs. 423 and 424 show one method of securing a cast-iron cap on a tall chimney shaft. The segments, being bolted together, keep in place by their own weight, and may afterwards be filled in with brickwork in continuation of the shaft. The flanges of the capping are ¾ in. thick as indicated in Fig. 424.

Stability of Tall Chimney Shaft.

A tall chimney may fail (*a*) by overturning owing to the foundations yielding; (*b*) from pressure of wind causing a simple overbalancing; (*c*) from pressure of wind causing the bricks to crush on the leeward side; (*d*) from the destruction of the material from heat, moisture, etc.; (*e*) from lightning; (*f*) from earthquake. A tall chimney could not fail by sliding on a bed joint. The effective pressure of the wind against a circular chimney is ·7854 of its value against a plane surface of equal area. The centre of pressure, or resultant of the forces, should fall within the middle third of the diameter of base, and the pressure on the outer edge should not exceed ten tons per square foot. These are common rules, but the precise circumstances should be considered.

Plumbing Chimney Shaft with Theodolite.

There are various ways of using a theodolite to ascertain how a chimney shaft deviates from the perpendicular. Find in which direction the chimney is upright by walking round it, say at a distance from the chimney equal to twice the height, then the direction of maximum inclination will be at right angles to this point. Set up a theodolite so as to get a view of the inclination (Fig. 425). Stretch a line on the ground at the base of the shaft (A B, Fig. 426), sight the top of the shaft and transfer the points *t t'* down on to the line A B, then sight the bottom of the shaft and transfer the points *b b'*; now bisect *b b'* and *t t'* in points *c c'*, and the distance from *c* to *c'* will be the amount the chimney is out of plumb. Another method would be to set up a theodolite, keeping the primary circle fixed in any one position throughout. Clamp and unclamp the vernier, and turn the tangent screw for adjustment as required to read the angle of both sides of the top and both sides of the bottom. Then the difference between the mean readings at the top and the

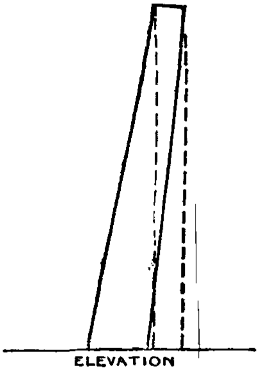


Fig. 425.

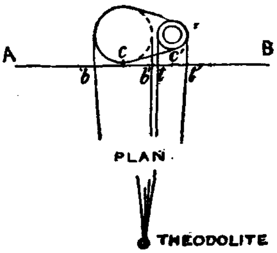


Fig. 426.

Figs. 425 and 426.—Plumbing Chimney Shaft with Theodolite.

mean readings at the bottom will be the amount the shaft is out of plumb in angular measurement. To convert this to feet, find the value from a table of natural tangents corresponding to these degrees and minutes, and multiply by the distance from the theodolite to the centre of the shaft. Neither of these methods will be mathematically true, but both of them will be as near as the result will be wanted.

Taking Down Tall Chimney Shaft.

Chimney-felling is risky work at the best of times. To ensure that a chimney shaft shall fall in a narrow compass it will be desirable to fix three guy ropes from the top, equally divided round the circle, and made fast at a distance from the base of the shaft at least equal to half the height. Openings should be cut in the brickwork of the base on opposite sides, and 9-in. by 9-in. studs inserted, about 4 ft. long, between 9-in. by 3-in. plates running through the thickness. Before making the openings, 9-in. by 3-in. raking shores both ways should be fixed at each corner of the base. Two openings in each side, with a brick pier left between, would, in the writer's opinion, be required; and when this is done, if there is no sign of cracking or settlement, and the studs are taking a good bearing, the intervening pier in centre of each side may be cut away. Everything must be done systematically, working at opposite sides in turn. Waste wood should then be piled round the base in sufficient quantity to ensure that the wood studs will be burnt through, and lighted at several points. A couple of look-out men during the operations should be posted sufficiently far off to command a view of the chimney from two directions at right angles, and near enough to warn the men if any signs of premature failure occur. Local circumstances and the construction and condition of the chimney stalk may render some variation on the above method desirable. A cheaper method, and one that would probably be satisfactory in the hands of an expert in explosives, would be to explode a small charge of dynamite in the bottom of the shaft, or to bore holes round the base and insert charges of gunpowder to be fired simultaneously.

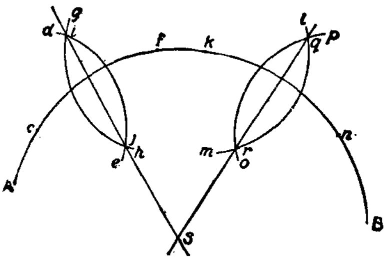


Fig. 427.—Finding Centre of Arch Curves.

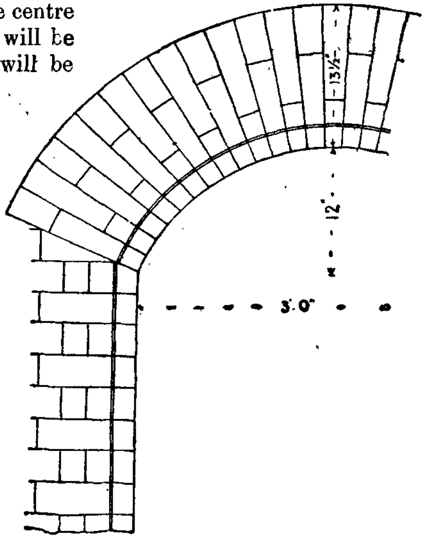


Fig. 428.

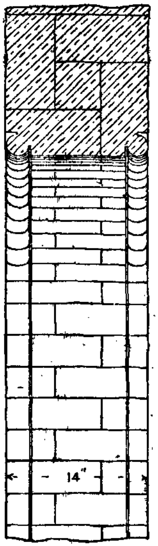


Fig. 429.

Figs. 428 and 429.—Moulded Segmental Arch.

Finding Centre of Arch Curves.

The centre points from which different arches are struck can be readily found by practical geometry, the most useful method of finding centres being that shown in Fig. 427. Let A B be part of the curve of any arch; from any point *c* with a radius *c d* strike the arc *d e*, and from any point *f* with the same radius strike the arc *g h* to intersect with the previous arc

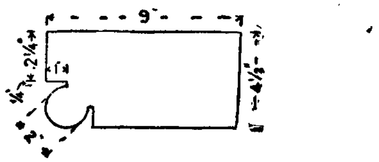


Fig. 430.—Moulded Brick for Segmental Arch.

at *i* and *j*, then a line drawn through *i j* will pass through the centre from which the arch curve was struck. Now take any point *k* and with any radius *k l* strike the arc *l m*, and from any point *n* with the same radius strike the arc *o p* intersecting the previous arc in *q* and *r*, then a line drawn through *q r* will intersect with the line drawn through *i j* at the point *s*, which will be the centre of the arch curve. When the arch has more than one centre, the various centres can be found in the same way, by taking different parts of the curve.

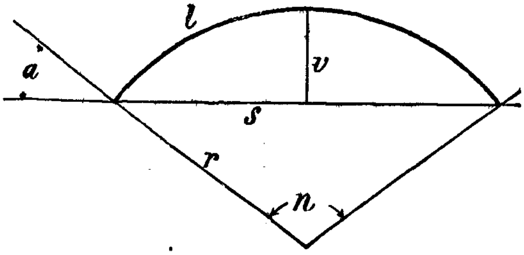


Fig. 431.—Setting Out Segmental Arch.

Terms Applied to Arches.

By an abutment is usually meant the side supports of a brick or stone bridge or arch of single span, or the last support on each side of a series of arches. A stop abutment is a wide pier at intervals of six or eight arches in a viaduct to limit the possible extent of collapse in the event of one of the arches failing. The term abutment is also used to express the meeting or abutting surfaces under pressure, as at the foot of a principal rafter. Voussoirs are the tapered stones or bricks used in forming an arch.

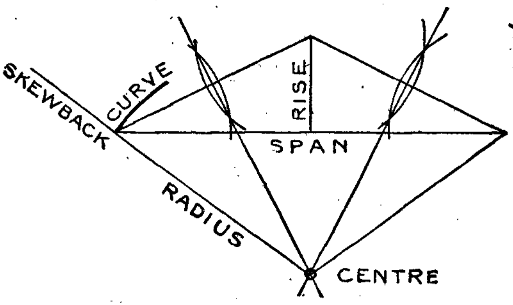


Fig. 432.—Setting Out Segmental Arch.

Moulded Segmental Arch.

The part elevation of a moulded segmental arch and reveals suitable for a 3-ft. opening, 14 in. on the face, 14-in. soffit, 12-in. rise, is presented by Fig. 428. Fig. 429 is a section, and Fig. 430 detail of the moulding The mitre between the arch and the reveal is not quite true, but the difference is so slight that it is of no practical consequence. With regard to finding, for a given rise and span, the skewbacks, the diameter of the whole circle of which the segment is a part, and also the exact measurement from skewback to skewback, it may be said that the relationship of the various parts of a segment arch is expressed by the following formula (see Fig. 431): *r* = radius of curve,

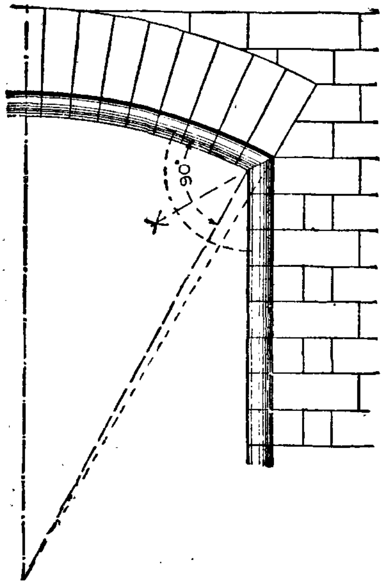


Fig. 433.—Bull-nose Segmental Arch.

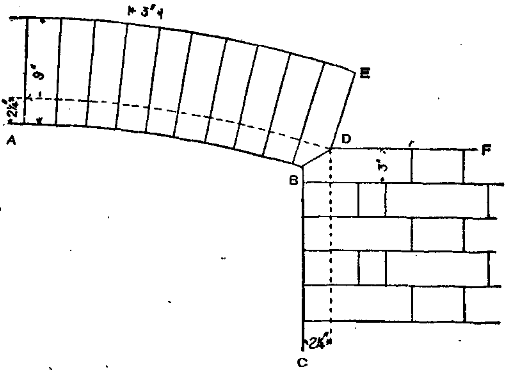


Fig 434.—Mitreing Bull-nosed Bricks of Glazed Arch.

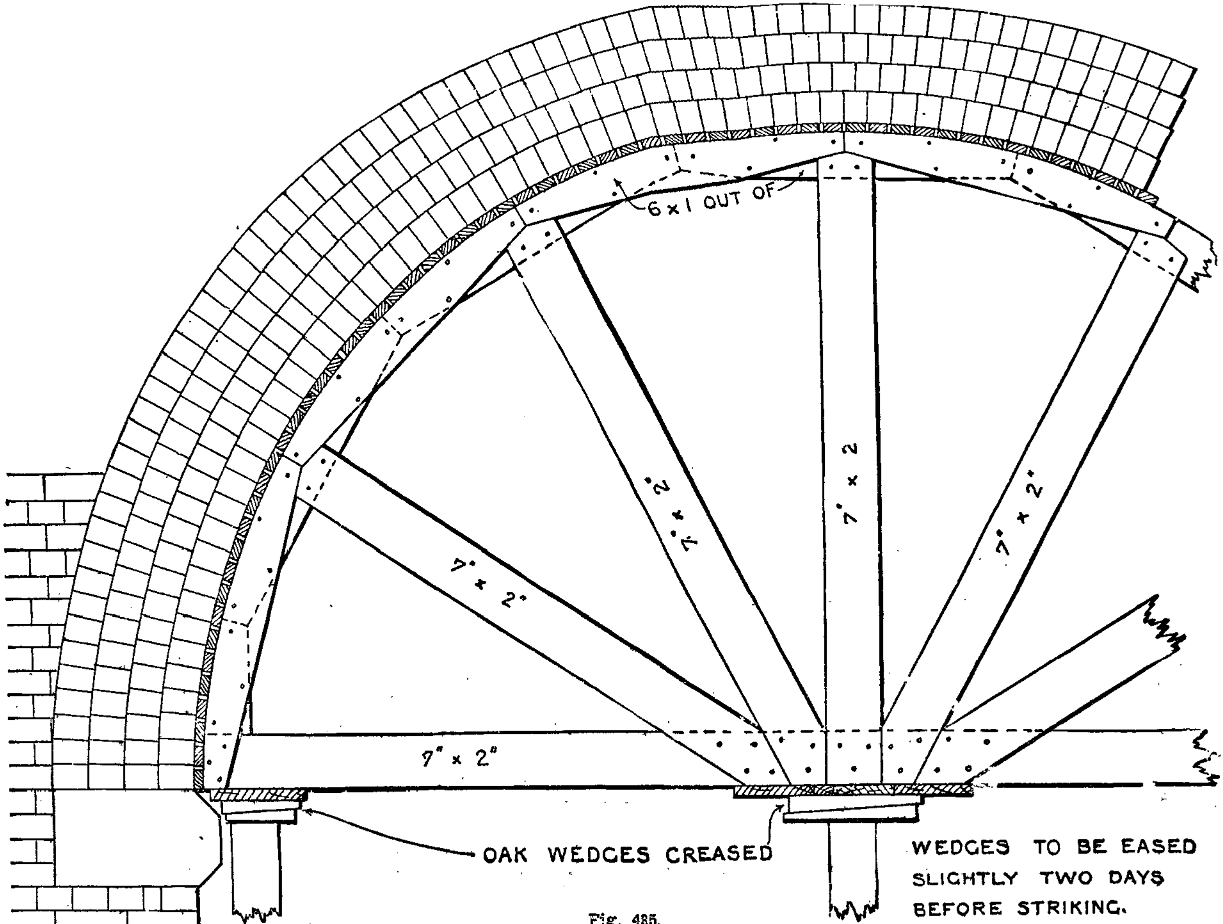

6 x 1 OUT OF
7" x 2"
7" x 2"
7" x 2
7" x 2"
7" x 2"
OAK WEDGES CREASED
WEDGES TO BE EASED
SLIGHTLY TWO DAYS
BEFORE STRIKING.

Fig. 435.

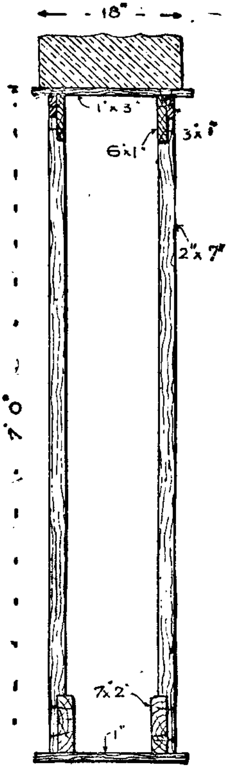

18"
7' 0"
1" x 3"
3" x 1"
6 x 1"
2" x 7"
7" x 2"
1"

Fig. 436.

Figs 435 and 436.—Semicircular Four-ring Arch and Centering.

v = versine or rise, s = chord or span, a = angle of skewback from horizontal, l = length of arc or soffit, n = number of degrees in centre angle, d = whole diameter of circle of which curve forms part, c = whole circumference of circle, $\pi = 3{\cdot}1416 = \frac{22}{7}$ = ratio of circumference to diameter, $l = \frac{\pi n r}{180} = \frac{1}{3}\left(8\sqrt{\frac{s^2}{4} + v^2} - s\right)$, $r = \frac{s^2}{8v} + \frac{v}{2}$, $s = \frac{r \sin n}{\frac{1}{2}(180 - \sin n)}$, $n = 360\frac{l}{2\pi r}$, $a = 90 - \frac{n}{2}$, $d = 2r$, $c = \pi d$.

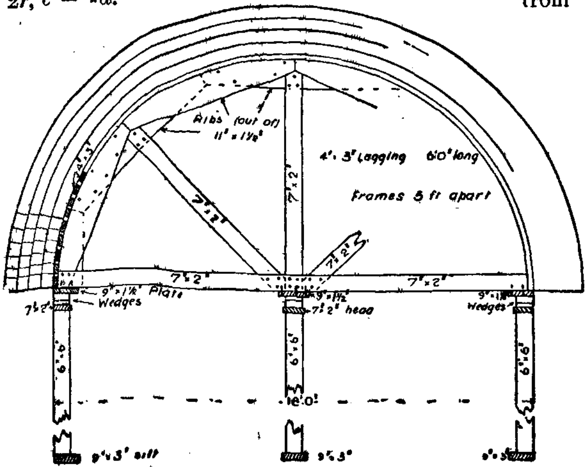


Fig. 437.—Centering for Arch of 18-ft. Span.

Example: Span 6 ft., rise 18 in.

$$\text{Radius} = \frac{6^2}{8 \times 1{\cdot}5} + \frac{1{\cdot}5}{2} = 3{\cdot}75 = 3 \text{ ft. } 9 \text{ in.}$$

Length of soffit

$$= \tfrac{1}{3}\left(8\sqrt{\frac{6^2}{4} + 1{\cdot}5^2} - s\right)$$

$$= \tfrac{1}{3}\left(8 \times \sqrt{11\tfrac{1}{4}} - 6\right) = \frac{(8 \times 3{\cdot}35) - 6}{3}$$

$$= \frac{26{\cdot}8 - 6}{3} = 6{\cdot}93, \text{ say } 7 \text{ ft.}$$

Centre angle

$$= \frac{360 \times 7}{2 \times 3{\cdot}1416 \times 3{\cdot}75} = \frac{2520}{23{\cdot}56} = 107 \text{ degrees.}$$

$$\text{Angle of skewback} = 90 - \frac{107}{2} = 36\tfrac{1}{2} \text{ degrees.}$$

Generally the span and the rise are given, and the simplest method of finding any other particulars is to put the span and rise down to scale and find by practical geometry the centre of a curve to pass through the three points as shown in Fig. 432.

Keystone of Arch not an Essential.

A keystone, although usual in the centre of many kinds of arches, is neither a theoretical nor structural necessity. Anyone who chooses to do so can build an arch without a keystone; the question is merely one of taste.

Skewback of Bull-nose and Segment Arch.

The skewback of a bull-nose segment arch, or other moulded arch to match jambs, will not fit if cut to a straight line, as the main part of the skewback must be radial to the curve of the arch; that is, it must point to the centre from which the curve is struck, while the moulded part must be mitred to bisect the angle at the springing, making two different bevels, as shown in Fig. 433. The moulding shown is the same as that in Fig. 429. The following is the proper method of cutting the skewback and mitreing the bull-nosed bricks of a glazed brick arch. Let A B C (Fig. 434) be part of the arch and jamb. Show the width of the bull-nose by a dotted line, and draw the mitre B D; from D set off the radial line D E in order to give the depth of the arch ring, and from D a horizontal line D F in order to give the top of the first brick; then complete in the ordinary way.

Semicircular Four-ring Arch.

Fig. 435 shows the part elevation of the head of an opening into a brick-built warehouse. It is a semicircular arch of four half-brick rings, and, as shown, it rests on its centering, all the details of which are given. Stone imposts, 21 in. by 12 in. on face, are also shown. The brickwork above and below the impost is in English bond. The manner in which the struts of the centering finish against the ribs is made clear by Fig. 436, which is a vertical section through the centering. A similar arrangement, shown in Fig. 437, was designed for an arch of 18-ft. span. Temporary diagonal bracing should be nailed on the posts. A semicircular arch should have about one half-brick ring for every 5 ft. span.

Another Style of Centering.

Figs. 438 and 439 give two views of a centering suitable for an arch of 45-ft. span, 18 ft. 6 in. rise, and 17 ft. 6 in. length. It is only strong enough to carry the arch bricks, and even then should be carefully put together, there being no surplus strength.

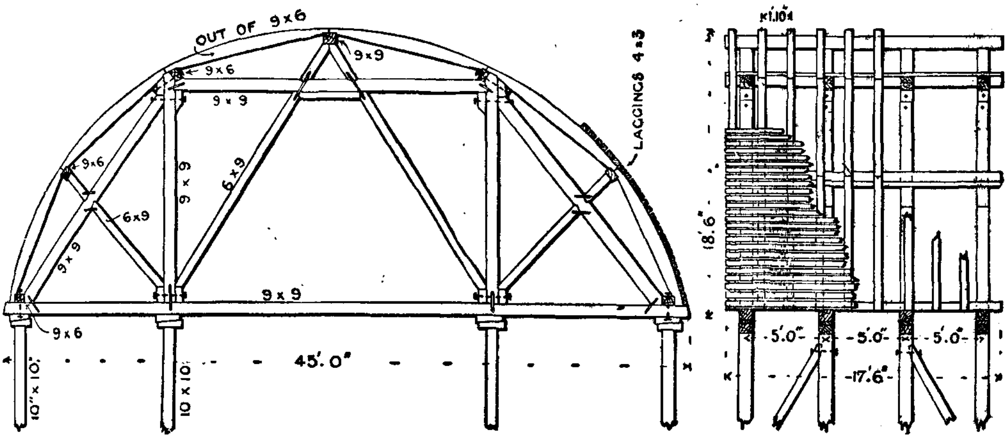

OUT OF 9×6
9×6
9×9
9×9
9×6
6×9
9×9
9×9
6×9
9×9
9×6
LACINGS 4×3
45'.0"
10"×10"
10×10
18'.6"
5'.0"
5'.0"
5'.0"
17'.6"

Fig. 438. Fig. 439.

Figs. 438 and 439.—Centering for Arch of 45-ft. Span.

Lacing Courses.

Lacing courses are a group of gauged bricks or bonding blocks set at intervals in a plain arch built in half-brick rings.

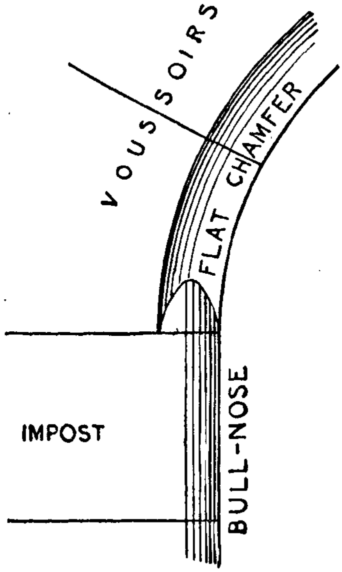

VOUSSOIRS
FLAT CHAMFER
IMPOST
BULL-NOSE

Fig. 440.—Junction of Arch and Quoin.

Also a course of ashlar work, coursed rubble, or brickwork built at intervals in a flint rubble wall to strengthen it. Lacing courses are so far from being universal in brick arch construction, that as a rule they are inserted only when directly specified or ordered by the clerk of works. In an arch that is on the point of falling, lacing courses may delay the event for a short time; but otherwise lacing courses have little effect on the strength or stability of the structure.

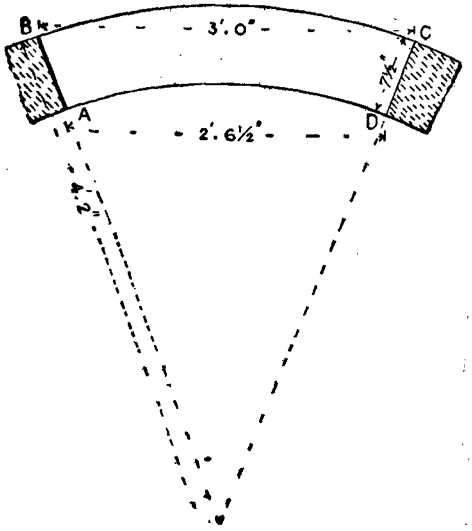

B
3'.0"
C
A
D
2'.6½"
7¼"

Fig. 441.—Template for Circle-on-Circle Brick Arch.

Junction of Arch and Quoin.

In a case where the opening for a doorway has a semicircular arch and the quoins for the sides are bull-nosed, while the arch is chamfered, the best form of junction will generally be settled by the architect, who sometimes has strong ideas upon such matters. It would probably produce the best effect if the bull-nose quoin were simply continued up in the same straight line to die off as it met the chamfer of the arch (see Fig. 440). The junction line is then the same as the outline of penetration between a cone and a cylinder. The cone would have a diameter of base equal to the width of the arch outside the chamfer, and the sides would incline at an angle of 45°. The cylinder would have a diameter of twice the radius of the bull-nose, and, to make a geometrical drawing, would be so placed that when the cone was standing on its base with the cylinder lying horizontally on the same plane the extreme penetration of the cone would cut through one-fourth of the circumference of cylinder, and a vertical line through the axis of cylinder would just touch the extremity of base of cone. To compare with the position in the actual arch, the cylinder would be vertical and the axis of cone horizontal.

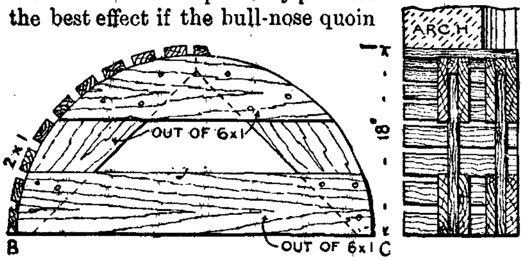


Fig. 442. Fig. 443.

Figs. 442 and 443.—Centering for Circle-on-Circle Arch.

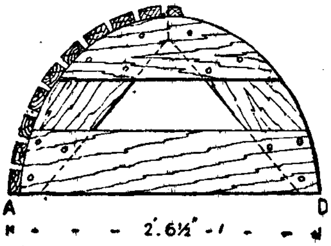


Fig. 444.—Elliptical Centering.

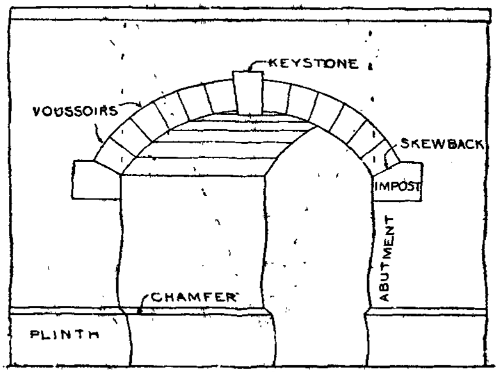


Fig. 445.—Skew Arch.

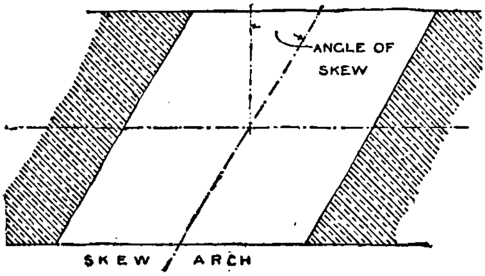


Fig. 446.—Plan of Skew Arch.

Circle-on-Circle Brick Arch.

The circle-on-circle is a false construction for a brick or stone arch. In wood such a construction may be allowable where the arch is a matter of form only, as in the framing over a pay-window in a railway booking office with bow front, but not where weight has to be carried. The reason is that where the versine to the chord of the curve in plan exceeds the thickness of the wall there is no place to receive the

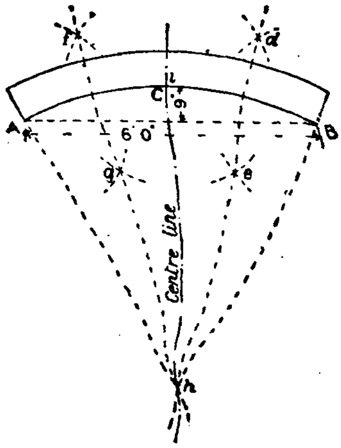


Fig. 447.—Setting Out Skewback.

thrust, and the pieces are held together by the jointing material or by ties. Where the wall is proportionately thicker there is a small amount of natural stability, but the writer would not care to use such an arch unless the thickness of wall were at least twice the versine in plan. A circle-on-circle arch should be adopted only in exceptional cases, such, for instance, as the masonry at the head of a certain design of bay window. In that case a semicircular arch of 3-ft. span is turned in a 7½-in. stone wall curved to a radius of 4 ft. 2 in. A template A B C D (Fig. 441) should be set out on the plan for the base of the centre, and the outside B C being a semicircle the framing may be set out as in Figs. 442 and 443, being flat on face, like an ordinary centre; allowance is made for the thickness of the laggings, which are 3 ft. over all. The back A D will be the same height (18 in.), but only 2 ft. 6½ in. wide, and therefore elliptical, as in Fig. 444.

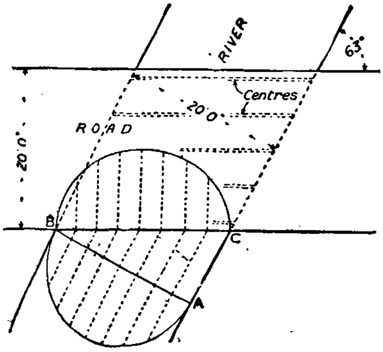


Fig. 448.—Skew Arch across River.

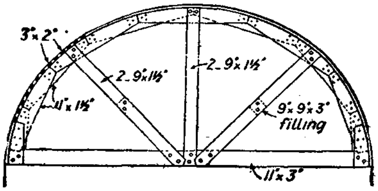


Fig. 449.—Centering for Skew Arch.

The curve of the ellipse may be set out by a mechanical method as fully described elsewhere in this work. The laggings, when nailed on, will form a flewing surface or spiral upon which the voussoirs or bricks may be laid. The supports for the centre would be the usual ones, the overhang not being sufficient to necessitate any exceptional course being adopted.

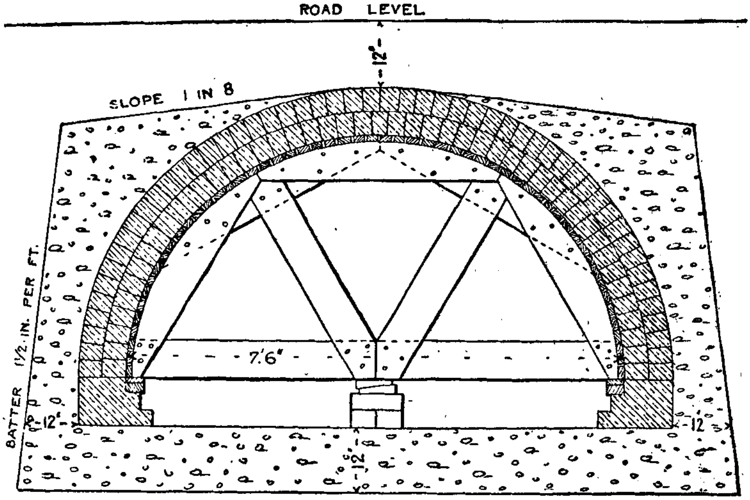


Fig. 450.—Brick Culvert on Skew.

Skewback of Arch.

A skew arch (Fig. 445 and 446) is one whose axis is not at right angles to its face, or whose two centre lines in plan are not perpendicular to each other. A skewback is the sloping surface against which the springing of an arch rests. Fig. 447 shows how the centre and direction of the skewback are found for an arch of 6-ft. span and 9-in. rise. Set off A B =

span, and point C on the centre line representing the rise. Then from C and B with any radius draw arcs intersecting in *d* and *e*; from A and C draw similar arcs intersecting in *f* and *g*. Produce *de*, *fg* to meet on the centre line, and this will be the centre for the curve of the arch which is to be struck with the radius *h i*. The skewbacks will be found by prolonging the directions *h* A, *h* B.

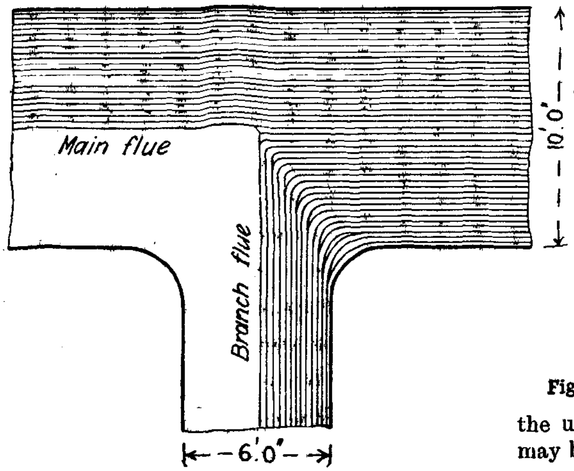


Fig. 451.—Arch at Junction of Furnace Flues.

Centering for Skew Arch.

The method of setting out a centre and framing for a skew arch crossing a river at the angle shown in Fig. 448 is described below. The plan of the road across the river being as in Fig. 448, the true elevation of the arch on face will be found as shown in that figure, where A B C is an angle of 90 − 63 = 27 degrees, and on A B is drawn the semicircular section of the arch across the river, with ordinates at any interval at right angles to A B and drawn through to B C. Then the heights of the ordinates to the semicircle on A B are transferred to B C to give the ellipse, which will be the true elevation of the face of the arch. This latter gives the outline for the centering, which may be made as Fig. 449. Five of these centres will be required, with 3-in. by 2-in. laggings running straight through.

Brick Culvert on Skew.

A semicircular arch over a brook crossing a public road could be built as suggested in Fig. 450. The arch is assumed to be 7 ft. 6 in. wide inside measure, and of 9-in. brickwork built at an angle. The centres may be 7 ft. or 8 ft. apart if necessary, and may be placed square to the centre line of the brook if carried through far enough for ends. This will avoid the use of elliptical centres, and the culvert may be built as a skew arch with spiral courses, or as an ordinary barrel vault with the end bricks cut on the rake, according to the class of work required.

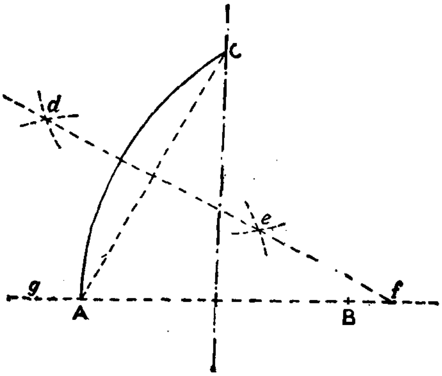


Fig. 452.—Obtaining Radius of Gothic Arch.

Arch at Junction of Furnace Flues.

A method of constructing an arch at the junction of furnace flues, the arch being continued round the curve, is shown by Fig. 451. A single ring of 4½-in. brickwork is hardly sufficient for a span of 6 ft., even when a very light load is carried, and both sides of a branch flue are not usually curved into the main flue, especially with so large a curve, as 2 ft. 6 in. radius. Generally the flue, when curved at all, is only curved in the direction of the current of the air in the main flue, so that both sides of the branch flue remain parallel. A flat arch gives greater thrust than an arch with more rise. If the arches are to have the same rise, and the main flue has a greater width than the branch flue, the arch will have a flatter radius. In these circumstances, if there were no curve in plan at the angles, the

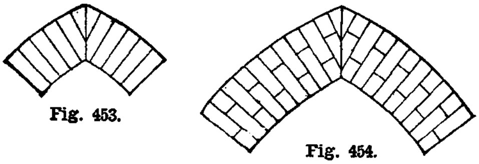


Fig. 453.

Fig. 454.

Figs. 453 and 454.—Straight Joint in Gothic Arch.

intersection of the two arches would give a slightly curved groin, which would, however, not give much trouble; but with the angles of the flue curved to so large a radius, the difficulty is greatly increased, and the method of working can hardly be indicated in any other way than that shown in Fig. 451, where the main flue is assumed to be 10 ft. wide. The thin lines indicate the courses of bricks in the arches, some being tapered courses; but the range of boilers must be enormous if so large a flue is required.

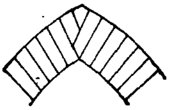


Fig. 455.

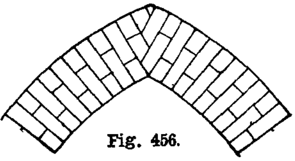


Fig. 456.

Figs. 455 and 456.—Alternate Bricks at Apex of Gothic Arch.

Gothic Arches.

When the semicircular Norman arches gave place to Early English the whole style of architecture underwent modification, and the heavy classic style, of which the Norman was more or less a copy, was replaced by pointed Gothic work. The windows were denominated "lancet" from their shape, being very narrow and pointed at the top. Then as time went on the windows were made wider and the arches became "equilateral." The same tendency to widen out resulted later in the "drop arch," which was still pointed but was wider than the equilateral arch. This soon became the Tudor or "four-centred arch," and ultimately the domestic Gothic had windows with mullions, transoms, and flat horizontal heads. An equilateral arch is therefore Gothic, but is only one of several varieties. The coloured plate, "Brickwork Arches," accompanying this work should be referred to, and will be found useful in this connection.

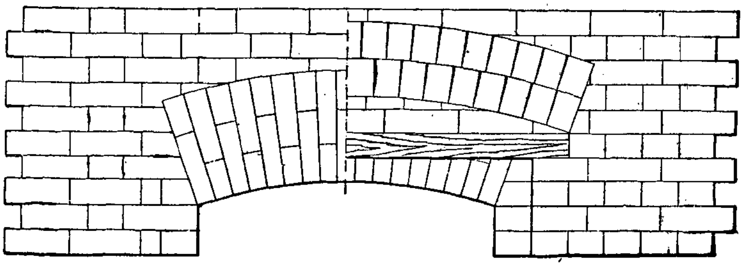


Fig. 457.—Gauged Camber Arch on Face of Wood Lintel and Common Arch.

Obtaining Radius of Gothic Arch.

The most practical method of obtaining the radius of a Gothic arch is as follows: Draw a horizontal line A B (Fig. 452) through the spring of the arch to represent the span; and draw a centre line setting off the height to the point of the arch C. Join A C, and with any radius greater than half A C draw the intersecting arcs *d* *e*, and produce through them a line to meet the line through A B in point *f*, which will be the centre of the arch curves from A to C. Then A g = B f will give the centre on the other side. The method of fixing the radius rod must be determined on the spot, and will vary according to the circumstances of the case.

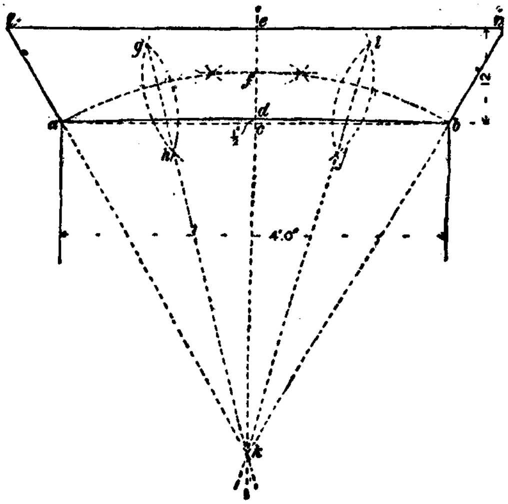


Fig. 458.—Setting Out Gauged Camber Arch.

Gauged Rubber Gothic Arch.

The objection to a gauged arch being formed of half-brick rings, apart from appearance, is that half-brick rings, not being bonded between the courses, are less able to withstand any settlement of foundations or spreading of piers, and so a long skewback and an arch keystone are proposed to reduce the length of arch without bond into smaller portions. Two methods are in common use for the point of a Gothic arch, and they are shown in Figs. 453 to 456 applied to arches of 9 in. and 12 in. on face.

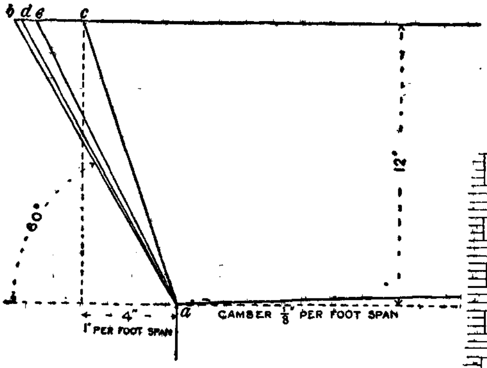


Fig. 459.

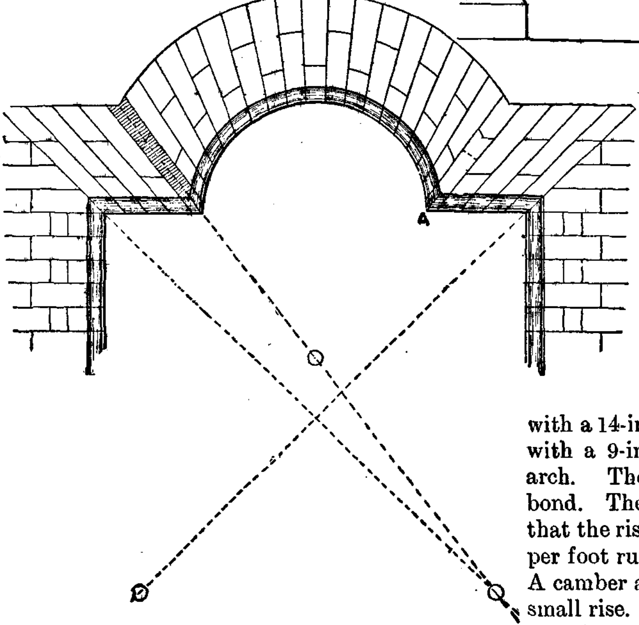


Fig. 461.

Among architects the straight joint (Figs. 453 and 454) is considered correct, but builders consider the alternate bricks at apex (Figs. 455 and 456) to be stronger.

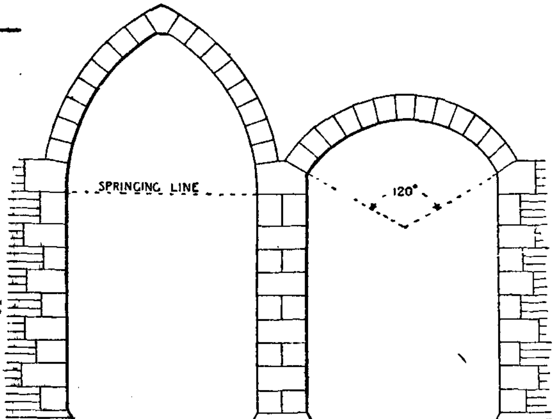


Fig. 460.

Fig. 459.—Alternative Methods of Setting Out Camber Arch.

Fig. 460.—Two Stone Arches Adjoining.

Fig. 461.—Venetian or Queen Anne Arch.

Gauged Camber Arch.

Fig. 457 is the external elevation of a window head in a 14-in. brick wall, finished externally with a 14-in. gauged camber arch, and internally with a 9-in. by 3-in. wood lintel and common arch. The brickwork is in single Flemish bond. There is a popular but false impression that the rise in a camber arch is limited to $\frac{1}{8}$ in. per foot run, but there is no hard and fast rule. A camber arch is defined as an arch having a small rise. A straight arch or square arch—namely, one having a flat top—is usually a cambered arch because of the small rise it has on the soffit, and some persons would limit the term to this kind, but that also would be incorrect.

Setting Out Camber Arch.

The following method of setting out a camber arch is that prevalent in the London neighbourhood: Set out opening, construct an inverted equilateral triangle, produce the sides of triangle past springing line to obtain skewbacks; give a rise of $\frac{1}{8}$ in. per foot from either side to centre; obtain joint lines in the usual way, then from these lines obtain the templates. Cut or specially moulded bricks would be used. In any lintel or straight arch, the angle of skewback should be such that the intersection of their directions will give a radius from the springing that would cause an arc to pass through the middle of the depth of the arch. The arch will then have the skewback as nearly perpendicular to the mean thrust as possible. Fig. 458 shows a method of obtaining this angle, the parts being lettered in order of construction. The following method has been advocated, but it makes no allowance for the possible variation of depth of arch: in the case of an arch of 4-ft. span set back from the jamb line a distance of 4 in., or 1 in. for every foot of span, and so obtain the skewback. Fig. 459 shows a comparison of three methods applied to the skewback of a 12-in. camber arch for a 4-ft. opening; line *b* is at 60 degrees; line *c* has 4-in. overhang; line *d* is the best position to receive thrust from the arch. Line *c* would, in the writer's opinion, cause too great a thrust on the abutments of the arch.

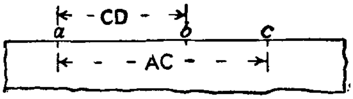


Fig. 463.—Trammel for Drawing Ellipse.

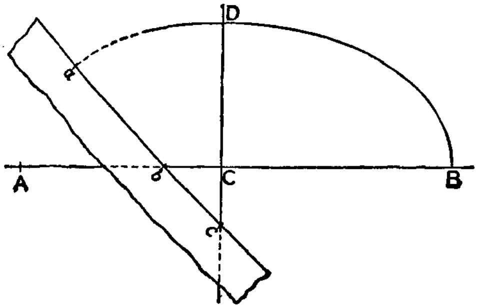


Fig. 462.—Drawing Semi-ellipse with Trammel.

Another method which might be adopted is to join the springing with the centre of extrados of arch and put the skewback at right angles to this line. This is shown by line *e* in Fig. 459.

Setting Out Venetian or Queen Anne Arch.

The Queen Anne arch occurs in the architecture of that period, and is also used over a Venetian window, which is a window of three lights, namely, a wide centre light with narrow side lights. The real joints are sometimes hidden by rubbing them over with a brick, false joints being then cut in and pointed. The centres from which the various curves and joints are struck are shown by small circles in Fig. 461. If there is a joint at the intersection of the semicircular part of the arch with the camber part, there will be a very ugly finish at the angle A owing to the difficulty of obtaining bricks that will

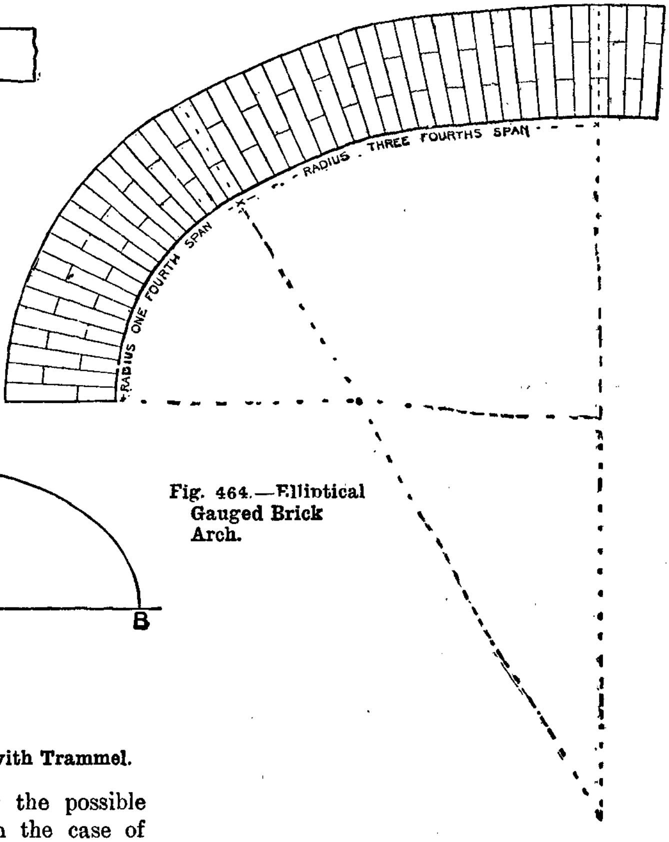


Fig. 464.—Elliptical Gauged Brick Arch.

keep such a sharp arris; the slightest chip on this arris will show a blotch of putty. It is therefore preferable to cut a mitre in the brick: see the shaded course in Fig. 461, half of which course finishes the camber part of the arch and half starts and forms the springing for the semicircular part, thus making a return from the camber to the semi on soffit of the shaded course, a bird's-mouth on extrados doing away with any joint in the angle. The principle of the mason's mitre—that is, cutting the return or intersection of two different planes in the one brick—should be adhered to wherever possible, as this method, as well as being easier and better for setting, is not so liable to be injured by the weather. A separate template will be required by which to cut the course corresponding to the shaded one shown in Fig. 461. Strike out the arch full size, and make a template the exact copy of the course corresponding to the shaded one in the illustration. Mark on this the mitre and two birds'-mouths, in the same position as on striking out. The stretcher should be always at the bottom, as a bat would be more liable to drop in this the weakest part of the arch. Cut the bricks to this template. If the stretcher will not work out to the length required to follow the line of cross-joints, it will have to be cut either out of a 12-in. brick or as shown by dotted line, and a piece stuck on with shellac, this joint being blinded and a putty joint being made in the proper position.

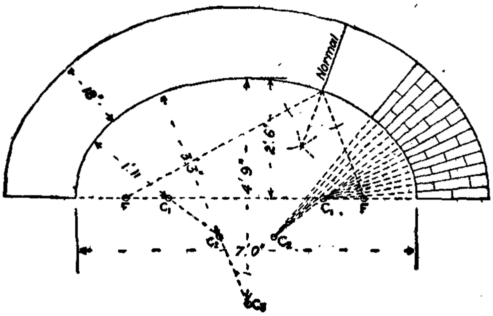


Fig. 465.—Setting Out Elliptical Gauged Arch.

Two Arches Adjoining.

Fig. 460 shows two adjoining openings in a brick building with stone dressings. The stone plinth is 16 in. high and is weathered at the top. An equilateral pointed arch is shown to the left, and a segmental arch of 120° to the right. Stone voussoirs are shown to both arches, 9 in. deep, springing from stone imposts.

Drawing Semi-ellipse with Trammel.

The best practical way of setting out a semi-ellipse without special instruments is by means of a trammel—of paper for a scale drawing, and of wood if full size. The method is as follows:—Draw two lines (Fig. 462) at right angles, to represent the axes or springing line and rise, and mark off the span A B and the rise C D. Then take a slip of paper (Fig. 463) with a clean-cut edge, and mark points *a b c* so that *a b* = C D and *a c* = A C. Then write against *a* "marking point," against *b* "springing line," and against *c* "centre line." Now use the slip with the letters against the corresponding lines to mark a series of points as shown on Fig. 462, through which a freehand curve should be neatly sketched, with the irregularities afterwards corrected by a French curve. The same principle is adopted in full-size work, straightedges being nailed down to give the springing, and centre lines and a lath used with French nails at *b* and *c*, and a pencil or scriber at *a*, so that the curve is drawn continuously.

Elliptical Gauged Brick Arch.

Fig. 464 represents to a scale of 1 in. = 1 ft., a little more than half of the front elevation of a 14-in. elliptical gauged brick arch over a 10-ft. span, showing the joints of the brick voussoirs. The method of getting the centres is shown in the figure. When struck from three centres as shown only two shapes are required for the bricks. If a true ellipse be used every brick in half the arch will be

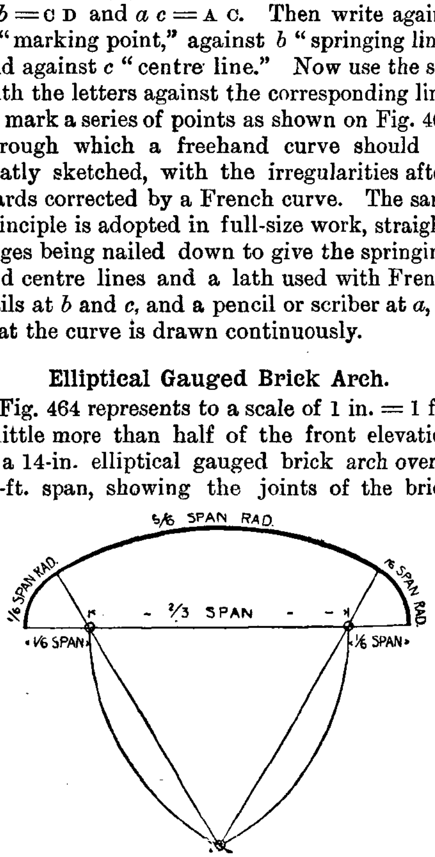


Fig. 466.—Setting Out Three-centered Arch.

different. The following shows how to obtain the joints of bricks for an elliptical gauged brick arch, the span of which is 7 ft. and the rise 2 ft. 6 in. Fig. 465 shows the arch in question, with a span of 7 ft. and a rise of 2 ft. 6 in. F and F are the two foci, and are obtained by taking the half span as a radius and the top of the soffit as a centre, and striking arcs on the springing line. Lines being drawn from the foci to any point on the soffit, and the angle made by them being bisected, the direction of a normal is given, which is the proper direction for the brickwork joint. For convenience in setting out the arch by centres instead of as a true ellipse, the requisite centres are marked C 1, C 2, C 3, and are respectively 1 ft. 11 in., 3 ft. 3 in., and 4 ft. 9 in. radius. Then the joints of the brickwork may radiate from the centres as shown. The exact place for C 2 may be found by striking an arc of 1 ft. 4 in. radius from C 1, and another of 1 ft. 6 in. radius from C 3; C 2 will be at the intersection.

Three-centered Arch.

"Three-centered arch" is a common term, and is derived from the fact that three different centre points are used for striking the arcs. It would popularly be called an elliptical arch, but although very much like one, it is not truly elliptical. The object of using three centres instead of a semi-ellipse is because the arch joints in the former case are very easy to set out, while in the latter case every joint would have to bisect the angle formed by lines drawn from the two foci. To find the points for striking the small arcs divide the span into six equal parts, the radius of the small arc being one part. Then open the compasses to the distance between these centres, and strike arcs intersecting below, giving the centre for the large arc, which will have a rise of a fraction over one-fourth of the span (see Fig. 466). The proportions may, of course, be varied to suit requirements.

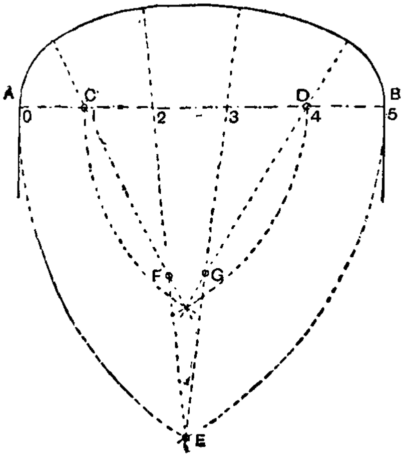


Fig. 467.—Setting Out Five-centered Arch.

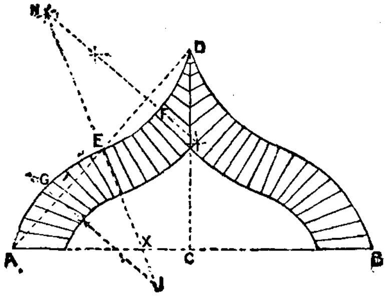


Fig. 468.—Ogee Arch.

Five-centered Arch.

For setting out a five-centered arch, draw the springing line of the arch of the required span A B (Fig. 467); divide this line into five equal parts, as numbered. With a radius equal to the span describe arcs intersecting at E, and from the intersection draw lines through each side of the central division. Then with a radius equal to three divisions draw arcs intersecting as shown, and from the intersection draw lines through the end of the next two divisions. The small circles show the centres for describing the five curves in the arch. Draw first from the centre C the curve at A up to the dotted line, then continue the curve from the centre F up to the next dotted line, and from E up to the next, then from G to the next, and from D to the finish.

Ogee Arch.

The ogee arch is not a form to be recommended in any material, least of all in brickwork; but the arch is sometimes used as an ornamental feature. The elevation (Fig. 468) and the following description are taken from Richards's "Bricklaying and Brickcutting." Let A B be out to out of extrados, and C D the rise of the same. Draw a line from A to D, and bisect it in E. Bisect D E, producing the centre line both above and below it, as in the segment, and the same with E A. Upon D E set up the rise upon that part of the centre line pointing to D B, and upon E A set up the rise upon the opposite side. Then describe the curves D F E, E G A, in the ordinary way. From the point H, by extending the compasses 9 in., put in that portion of the intrados from the line E I to the centre line C D, and from the point I, by decreasing the distances in the compasses 9 in., draw in the part of the intrados from E I to the base line A B. Deal with the bottom portion of the ogee as a scheme, by getting the shape of the template from the point X, made by E I cutting the base line; and the top part as a segment, obtaining the template from the point H. Traverse the templates, accurately fill in the courses, and mark the bevels and lengths.

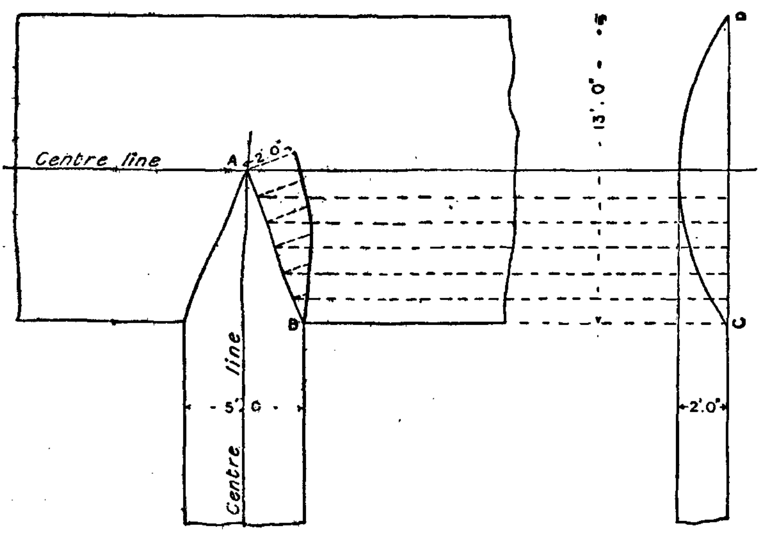


Fig. 469.—Groin in Brick Arch.

"Axed" Arch.

An "axed arch" is an arch where the bricks are roughly axed or cut to a wedge-shape so as to make a parallel and closer joint than a plain arch where the bricks are uncut, but not so close or true as gauged work, where the bricks are cut and rubbed. A rough axed relieving arch is illustrated in the coloured plate "Brick Arches" accompanying this work.

Groin for Brick Arches.

Fig. 469 shows how to set off a groin in a brick arch; size, 5-ft. span, with 2 ft. rise into a 13-ft. span with 2 ft. rise. A B is a plan of the groin, C D a section of the main vault. Take any divisions on A B and project them on to C D, then the heights from C D to the soffit of the curve will give the heights perpendicular to A B for the curve of the groin.

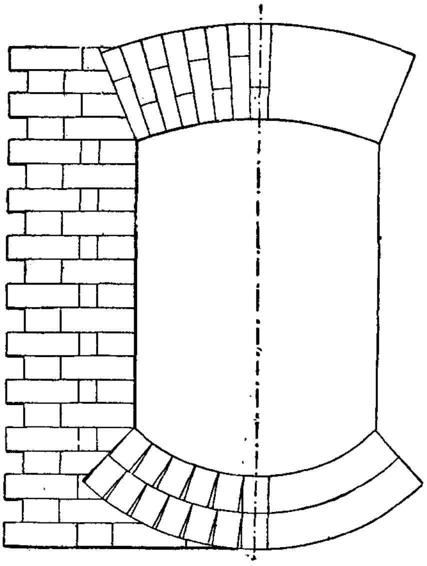


Fig. 470.—Opening with Arch Top and Bottom.

Openings in Brickwork.

Openings in brickwork for doors and windows on the ground floor are required to be central and symmetrical with regard to the passages and rooms. They are generally arranged to have a width equal to a given number of brick lengths, and any intervening piers are made multiples of a brick or half-brick in width. The openings on the upper floors should, as a rule, be directly over those below, which arrangement is called building void over void. An arch must be turned over an opening to carry the weight of the brickwork above and the loads resting on the brickwork. Over internal openings a relieving arch is generally turned in one, two, or more half-brick rings, according to the span, and generally 9 in. wider than the opening, to permit it to stand clear of the lintel which supports the brick core under the arch and gives a horizontal head to the opening. In external walls the outer 4½ in. is generally carried by a gauged arch having only a slight camber and hiding the rough relieving arch behind.

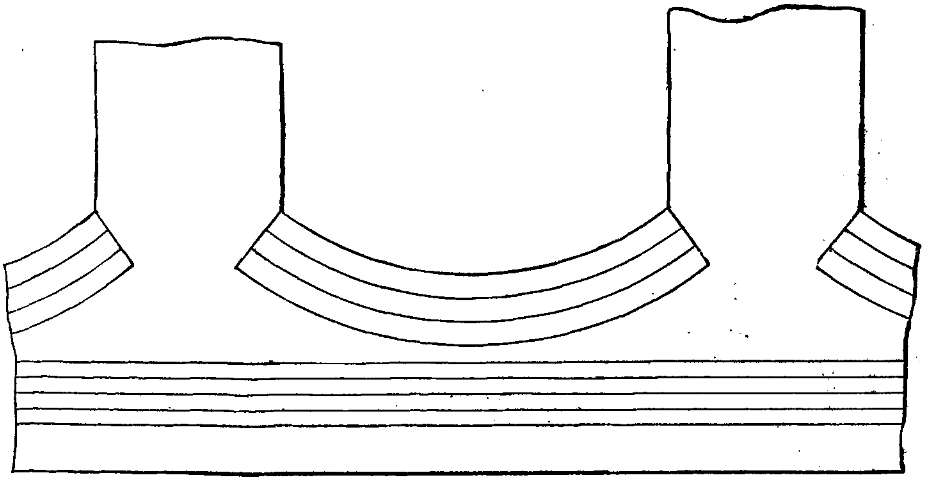


Fig. 471.—Inverted Arch.

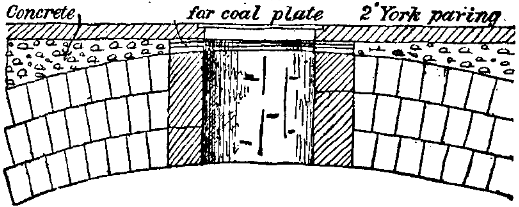


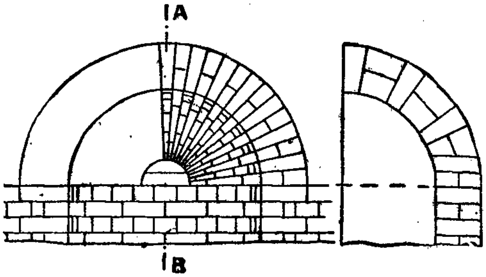
Fig. 472.—Coal-hole in Brick Arch.

Opening with Arches Top and Bottom.

Fig. 470 shows a 2 ft. 6 in. by 3 ft. opening in the basement of a brick building. There is at the top a gauged camber arch 12 in. deep, and at the bottom an inverted arch of two half-brick rings. The wall is built in Flemish bond.

Inverted Arches.

When the soil upon which the building rests is inadequate for the support of isolated loads, inverted arches should be constructed under openings or between piers. These arches serve to distribute the pressure over a continuous surface and reduce its intensity: for example, openings in piers of railway viaducts should always have inverts, and the columns in large churches are frequently connected below the



Figs. 473 and 474.—Brick Niche.

floor line by inverted arches and continuous footings. The mode of action and the calculations are precisely the same as for an ordinary arch; but the load and reactions are reversed; that is, the position of the load in the one case is taken by the reactions in the other, and *vice versâ.*

Forming Coal-Hole in Brick Arch.

A sectional view showing the formation of a coal-hole through a 1½-brick arch with pavement over is presented by Fig. 472. To form the coal-hole, a circular centre is used, hung by a rope handle with a crossbar for lifting out when the work is finished. This centre is stood on the arch centering where the opening is required, and when the arch is built up near to it the ring for the coal-hole is put in, and the arch built round it.

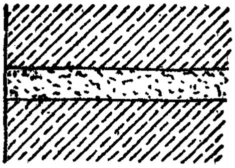


Fig. 475.—Flat or Flush Joint.

Brick Niche.

An elevation of a niche in brickwork is presented by Fig. 473, and a cross section showing the bonding by Fig. 474.

Pointing for Brickwork.

Flat or flush joints (Fig. 475) have the mortar pressed flat with the trowel while building; they are suitable for walls to be limewashed or distempered.

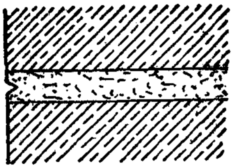


Fig. 476.—Flat Joint Jointed.

Flat joints jointed (Fig. 476) are similar to flat joints, but an iron tool called a jointer or the edge of a trowel is drawn along the centre against a straightedge to give an increased appearance of regularity in the work.

Keyed joints (Fig. 477).—In these the depression is broader, shallower, and curved, but is otherwise the same as a flat joint. In double-jointing a line is cut on each edge of the mortar next to the bricks.

Struck joint as usually done is shown in Fig. 478, but this should only be allowed in overhand work. It should be formed as shown in Fig. 479 to prevent lodgment of water, the lower edge being cut off against a straightedge. This joint is usually made as the work proceeds.

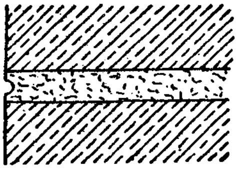


Fig. 477.—Keyed Joint.

Weather joint or V joint pointing, used principally by masons (Fig. 480), is very good in the bed or course joints, and Fig. 481 in the vertical joints of brickwork. These joints are generally made by raking out the mortar to a depth of ½ in. or ¾ in. before it has set, and pointing just before removal of scaffolding, working downwards from top of wall. Called jointing if done as work proceeds, pointing if done afterwards.

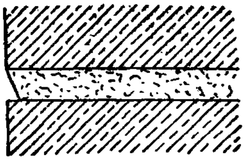


Fig. 478.—Overhand Struck Joint.

Blue or black pointing is generally used with red brickwork, and is made by mixing the mortar with fine ashes instead of sand to give a dark colour.

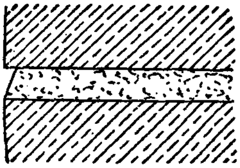


Fig. 479.—Struck Joint.

Tuck pointing (Figs. 482 and 483) is used for new or old work, but more often for the latter. The joints are raked out and filled flush with coloured mortar to match the brickwork, or rubbed over with a piece of soft brick of similar colour. A narrow groove is then run along the centre of each joint by the trowel, and the mortar allowed to set. The material for tuck pointing is then prepared, viz. pure white lime putty, with silver sand or marble dust in it, but not plaster-of-Paris, which is too soluble. The mortar having set, the groove is filled with the white putty, which is allowed to project beyond the face of the wall and is pressed firmly into the groove; it is then cut to a parallel line above and below about three-sixteenths of an inch wide, by a "Frenchman," leaving the appearance of perfect bricks with a thin uniform joint.

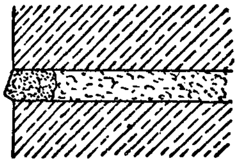


Fig. 480.—Weather Joint.

Bastard or half-tuck pointing (Fig. 484) the ridge or fillet, instead of being of white lime putty, is of the same material as the stopping.

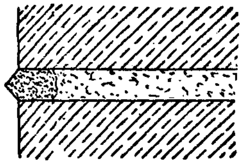


Fig. 481.—Weather Joint.

Effect of Frost.—Pointing must not be done during frost, as the expansion of the moisture in it by freezing would tend to throw out the pointing and disintegrate the joint. In ordinary circumstances "a neat struck joint as the work proceeds" is the best and soundest method of making the joint, and in frosty weather no brickwork should be allowed to proceed. Usually, the bricklayers continue work, and cover the top course with boards when they leave for the day; the joints are afterwards raked out and pointed in milder weather.

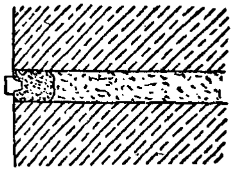


Fig. 482.—Tuck Pointing.

Specification for Re-pointing.—When a brick building is to be re-pointed with Portland cement mortar the following specification will be found suitable. The contractor is to provide all tools, water, scaffolding, plant, and labour for carrying out this contract. The brickwork is to be scraped clean, well washed and rubbed over with a brick of good colour. All the mortar joints are to be raked out carefully to a depth of ¾ in., and pointed in cement with a weathered joint to sketch (Fig. 480), struck in regular lines the whole width of the joint. The pointing stuff is to be composed of 1 part of approved Portland cement to 2 parts clean sharp pit sand mixed in small quantities and used immediately. The brickwork to be well wetted in advance of the pointing. All putlog holes to be filled up to match the other work, and all rubbish to be removed, leaving all clean and perfect on completion.

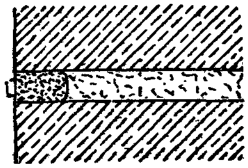


Fig. 483.—Tuck Pointing.

Stability of Walls.

In the case of a boundary wall, wind pressure is the active or upsetting force, and the weight of the wall is the passive force, or resistance; for equilibrium, the moments (that is, the forces multiplied by their leverages) must be equal; 1 ft. of the length of the wall is taken for comparison. The wind, acting in parallel lines at right angles to the wall and equally over the surface, may be assumed to be collected at half the height of the wall, and to act with a leverage from that point to the ground line. The wall resists by its weight, which may be assumed to be collected at its centre of gravity, and as this is in the middle of the thickness of the wall, overturning must take

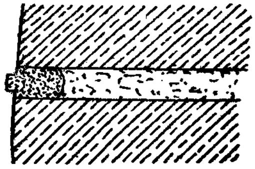


Fig. 484.—Half-tuck Pointing.

place, if at all, at the ground line on the outer face of the wall. The resistance has a leverage of half the thickness of the wall. In the case of a retaining wall, failure tends to take place by the sliding downwards of a wedge of earth behind it. This wedge is measured as follows: Every soil is supposed to have a natural slope; thus, if an excavation is made in any soil, the sides of the excavation, if left unsupported, would gradually fall away until a permanent slope was attained; or a heap of material left to itself would ultimately have sides of a certain slope. This slope will be the angle at which this soil will remain at rest. Suppose the slope to be at an angle of 45°, then a retaining wall would have to hold up the wedge of earth between this slope and the back of the wall; but it is found in practice that when a wall gives way this wedge of earth does not all fall away at once; it divides into two equal parts, the half next the wall falling at once and the remainder in the course of years; so that in calculating the stability of the wall allowance would be made for a wedge of earth of 22½°, this being the half that falls away, separating from the other part at what is called the line of rupture. The calculation is made on the ordinary principles of mechanics, just as if the wedge were a loose piece sliding without friction. The effect is the same as if the weight of the wedge had to be supported at a point one-third of the height up the back of the retaining wall, where the line of thrust is produced in a horizontal direction. The combination, by parallelogram of forces, of this thrust with the weight of the wall acting through its centre of gravity produces the resultant of thrust in the brickwork, which should, as a rule, fall within the middle third of its thickness. Calculations for determining the requisite thickness of walls under various conditions of soil and surroundings are given later in this work.

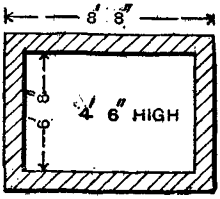


Fig. 485.—Diagram of Rectangular Building.

Pillars: Relation of Height to Strength.

The strength of pillars of stone or brick begins to decrease sensibly when the height exceeds six times the least diameter or width. Nominally, the safe load may be $\frac{1}{10}$ the ultimate load, or $\frac{1}{3}$ of the load at which fracture may commence, but practically much lower values are taken for the working load, owing to the unusual precautions taken to reach a high figure in testing and the large contingencies that arise in practice. The following table gives the writer's practice, height not exceeding six times least thickness :—

Material.	Safe load tons per ft. sup.	Ratio of working load to ultimate strength.
Granite	15	1 per cent.
Sandstone and best Portland ...	12	2 ,,
Limestone (ordinary)	9	2½ ,,
With stone template interposed :—		
Blue brick in cement	9	5 ,,
Stock brick in cement	6	5 ,,
Stock brick, lias mortar ...	5	10 ,,
Stock brick, grey lime mortar...	4	25 ,,

Maximum safe dead load in buildings not subject to vibration = 1½ times the above.

Approximate rule to allow for varying heights of columns and piers :—

W = tons per ft. super., as given in table.

r = ratio height of pier to least thickness.

$$\text{Safe load tons per square foot} = \frac{24\,W - Wr}{18}$$

Wall for Swimming Bath.

The cement concrete wall at the 6 ft. deep end of a swimming bath measuring 60 ft. by 25 ft. may be 1 ft. thick at the top and 2 ft. 6 in. at the bottom, built with a straight batter at the back. The thickness of the side walls is constant for any given level, the rise of the bottom towards the shallow end (3 ft.) merely cutting off the bottom portion of the wall.

"Settling" of Buildings.

When cracks appear in the main walls of a building, especially if partially horizontal, the building is said to have "settled." This use of the word is not strictly correct, and the subsidence of the brickwork is not always the primary cause of the cracks. The origin lies more often in the soil, and the "settlement" may arise from various causes, such as the consolidation of made ground, the displacement of sand by the action of water, and the shrinkage of clay, due to evaporation of the moisture in it. Sometimes the soil remains firm, and the "settlement" is confined to a portion of the building where the foundations are carried to a greater depth than for the adjoining parts, and then the increased number of mortar joints causes this portion to sink more than those adjoining. When a building is erected on peat a gradual settlement of the whole mass may occur, but, if the work has been properly carried out, there is no harm in this and no damage is done. Example:—Engine, boiler, and accumulator house, built of stone on peat, at the Alexandra Docks, Newport, where the working of the accumulator puts the whole into a state of vibration which can be felt at a distance of many yards. In one case in which the writer was called in to report upon "a settlement of the building," he found that a partition wall had been built on an 11-in. by 4-in. joist under the kitchen ceiling, and the shrinkage of the timber had caused the partition to crack and separate from the main wall. In another case the shrinkage of the clay in a dry summer caused several of the external walls of the houses in one district to so crack that they had to be taken down and rebuilt. In another case the bursting of a water main washed the sand from under the foundations of a building, which necessitated pulling down and re-erecting. The strict use of the word "settled" would confine it to the consolidation of the ground and of the mortar joints, etc., by the weight of the superstructure, so that the whole has reached a settled or permanent condition, but it is not often so limited in its application. This kind of settlement, which occurs more or less in all buildings, produces no bad effect, but unequal settlements cause cracks in the walls, and often lead to other and more serious damage.

Measuring Brickwork.

The method of measuring brickwork used in the south of England is briefly as follows:

All brickwork being reduced to a standard of 9-in. work and priced at per square yard or per rood of 7 square yards, the following rules will apply:

Ft. sup. 4½ in. divided by 18 = square yards reduced.

Ft. sup. 9 in. divided by 9 = square yards reduced.

Ft. sup. 13½ in. divided by 6 = square yards reduced.

Ft. sup. 18 in. divided by 4½ = square yards reduced.

Square yards reduced divided by 7 = local roods.

Square yards reduced divided by 4 = cube yards.

The standard footings can be measured in equivalent feet super. of the wall that the footings are under, namely, 4½ in. = ½ ft., 9 in. = ⅞ ft., 13½ in. = 1¼ ft., 18 in. = 1⅝ ft. super. per ft. run. Chimney breasts can be taken as solid wall, equal to the projection, and reduced as above. For joists, rafters, etc., multiply the breadth by the thickness both in inches and by the length in feet, and then divide by 12 and again by 12 in order to get the result in cubic feet. In order to measure the quantity of brickwork in a rectangular building, as shown in Fig. 485, for any thickness take the length out to out, add the length to the width in the clear, double the sum, and then multiply by the height. In the case shown by Fig. 485, 8 ft. 8 in. + 6 ft. 8 in. = 15 ft. 4 in.; 15 ft. 4 in. × 2 = 30 ft. 8 in.; 30 ft. 8 in. × 4½ = 138 ft. sup. of 9-in. brickwork. Assuming that a brick and its joints make the working dimensions 9 in. by 4½ in. by 3 in., each brick with its joints will occupy $\frac{3}{8} \times \frac{1}{4} = \frac{3}{32}$ ft. sup., and the total number of bricks in 138 ft. sup. will be $\frac{138 \times 32}{3} = 1472$, say 1,500 bricks without footings. If the walls are to have the usual footings (in addition to the height) of 4 ft. 6 in., that will be two courses, 14 in. and 18 in., or an average width of 16 in. by the same length as before, namely, 30 ft. 8 in. and a thickness of 6 in., making 20½ cub. ft. Each brick will occupy $\frac{9 \times 4\frac{1}{2} \times 3}{1728}$ cub. ft., so that 20½ cub. ft. will contain $\frac{20\frac{1}{2} \times 1728}{9 \times 4\frac{1}{2} \times 3} = 292$, or say 300 bricks (to allow for waste), or 1,800 bricks in all.

Measuring Brick Footings.

The correct method of measuring and entering brick footings is as follows, say for a two-brick wall:—

126·9 1·0	126·9	3¼ b. foots.

Top course 2½ b.
Bottom course 4 b.
2)6½
Average 3¼ b.

Then on the abstract the footings will be entered under the usual columns thus:—
Reduced brickwork in mortar—

	1 b.	1½ b.	*Ddt.*	
			1 b.	1½ b.
	31·9	253·6		
⅓ =	10·7	21·2		
	21·2	274·8		

= 1 rod 3 ft.

In the above the 126·9 is taken twice and entered in the 1½-b. column for 3 b. of the footings. For the odd ¼ b. the 126·9 is divided by 4 and entered under the 1-b. column, and ⅓ b. deducted to give the value in 1½ b., which is then transferred to the 1½-b. column. For 20-ft. run of 18-in. wall, with double course at bottom, the items will stand thus on the dimension sheet:—

20·0 1·0	20·0	3¼ b. footings	2½ 3
20·0			3½ 4
—·3	5·0	4 b. do.	4) 13
			3¼ average

And on the abstract sheet thus:—

	1 b.	1½ b.	Deduct.	
	5·0	40·0	1 b.	1½ b.
	20·0	16·8		
⅔	25·0	56·8		
	16·8			

And on the fair bill thus:—"56 ft. 8 in. sup. reduced brickwork in footings." On the abstract the 20 ft. of 3¼-b. footings will be entered under the 1½-b. column twice for the 3-b. and then ¼ of the amount will be entered in the 1-b. column for the ¼ b. The 5 ft. of 4-b. footings will be entered four times under the 1-b. column. The 1-b. column will then be cast, multiplied by ⅔, and the product entered under the 1½-b. column, which will then be totalled up and transferred to the bill.

For a 9-in. wall with a double course at the bottom, the quantities for 20-ft. run will stand on the dimension sheet in the manner shown below.

ft. in.	*ft. in.*		
20 0 0 6	10 0	1¾ brick footings	1½ 2
20 0			2)3½
0 3	5 0	2 brick footings	1¾ average.

And on the abstract sheet as follows:

	1 brick.	1½ brick.	Deduct.	
	ft. in.	*ft. in.*	1 brick.	1 brick.
	10 0	5 0		
	10 0	13 4		
⅔	20 0	18 4		
	13 4			

And on the fair bill thus: "18 ft. 4 in. sup. reduced brickwork in footings." The abstract figures are obtained as follows: 10 ft. of 1¾-brick footings will be entered as 10 ft. under the 1-brick heading for the 1 brick, and 5 ft. under the 1½-brick heading for the ¾ brick. The next item will be 5 ft. of 2-brick footings entered as 10 ft. under the 1-brick heading. This being totalled up, ⅔ is taken for the equivalent in 1½ brick, and transferred to the 1½-brick column. The 1½-brick column is then totalled up for the amount in the bill. The concrete usually projects 6 in. on each side of the footings, as that allowance has to be given in the excavation to give the bricklayer room to work.

Measuring Brick Piers.

With regard to measuring up a 14-in. pier that projects 4½ in. from a 9-in. wall, if the pier is in a wall faced with a better quality of brick than in the heart of the wall, the measurement of length for the extra cost of facing includes the whole of the exposed surface, making the allowance for each pier 9 in. beyond the net length of the wall; but if the wall is of the same bricks throughout, no extra measurement is allowed. If the brickwork in the bill of quantities is described as "... rods sup. reduced brickwork in mortar finished with neat struck joint as the work proceeds," there will be no other extra in connection with the piers; but if the face of the wall is pointed, then the same allowance of 9 in. beyond the net length of the wall will be made for each pier in measuring the pointing. If the tops of the piers are tumbled-in to the wall, they should be numbered and described as "No. ... tumbling-in to top of 14-in. piers, including cutting and waste."

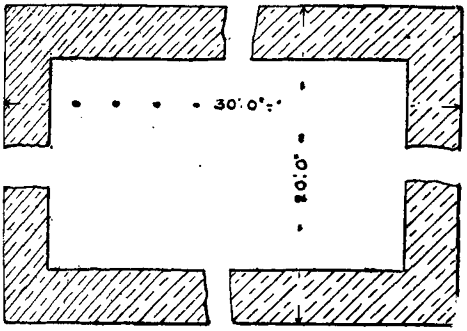


Fig. 486.—Plan of Building for which Quantities are to be taken off.

Quantity Surveyor's System of Measuring Brickwork.

The Dimension Sheet in the next column, corrected and annotated from Fletcher's "Quantities," shows the quantity surveyor's method of measuring brickwork. The building is as shown in plan (Fig. 486) and in section (Fig. 487).

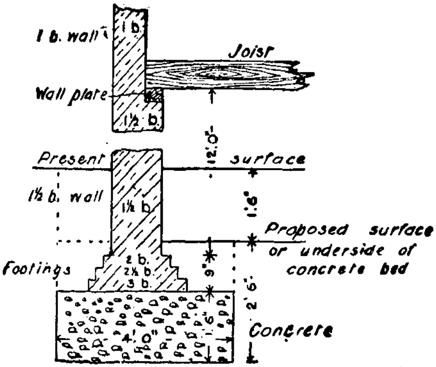


Fig. 487.—Section of Wall, etc., for which Quantities are to be taken off.

DIMENSION SHEET.

	ft. in.	ft. in.	
	33 0 23 0 1 6	1138 0	ft. in. 30 0 wall. 1 1½ ftgs. (half each end). 1 9 concrete (do.). 32 10½ Dig, wheel, and deposit to surface.
	95 6 4 0 2 6	955 0	20 0 (width of building.) 2/1½ b. = 2 3 (thickness). 17 9 (net length end walls). ft. in. 30 0 front wall (gross), *i.e.* out to out. 17 9 end wall (net), *i.e.* in the clear. 2/ 47 9 95 6 total length four walls. Ditto to trenches, part return, wheel, fill, and ram (walls collected).
	95 6 4 0 1 6	573 0	Concrete as described.
	95 6 9	71 8	2½ b. footings.
	95 6 12 0	1146 0	1½ b. wall to top of plate (ground floor).
	97 0 18 0	1746 0	1 b. add to roof plate. (*Note.*—4½ in. set-off on inside will add 1'6" to length of four walls.)
	3 6 7 0	24 6	1 b. ddt. entr. dr. } Revl. 4½" each side 3" head
	4 3 7 3	31 0	½ b. " " "
3/	3 0 6 0	54 0	½ b. ddt. wdws. (gd. flr.) } Do.
3/	3 9 6 3	70 4	1 b. " " " "
2/3/	2 6 5 6	82 6	½ b. " (1st & 2nd flr.) } Do.
2/3/	3 3 5 9	112 2	½ b. " " " "

Measuring Hollow or Cavity Walls.

One rule for measuring hollow walls is as follows :—Measure them solid as reduced brickwork, state the thickness, make no deduction for the cavity, but state its width, the kind of ties used, the number of ties to each superficial yard, and whether hay bands or movable boards are placed along the hollow to keep out falling rubbish. Another rule is not to bring the work to standard thickness as reduced brickwork, but to bill the work as feet super. with description, thus :—

rods.	*ft.*	*in.*		
	6257		supl.	Hollow wall of two thicknesses of brickwork 9 in. and 4½ in. respectively, with 2¼ in. cavity, bonded with galvanised wrought-iron wall ties, four to each superficial yard, and weighing 60 lb. per hundred, and allow for keeping the hollow clear of droppings of mortar and rubbish, and for leaving openings at bottom of hollow and for cleaning out hollow and filling up opening at completion.

Pricing Brickwork.

It being assumed that bricks (9″ × 4¼″ × 3″) cost 35s. a thousand, Portland cement 3s. 6d. a bushel, and sand 2½d. a bushel (all as delivered on the works), the following shows how to find the cost of these materials for a cubic yard of two-brick wall (mortar, 1 of cement to 3 of sand, joints to show ½″ thick). There are 21 bushels in a cubic yard. For Flemish bond a unit of size would contain 6 bricks and give

$$
\begin{aligned}
18\tfrac{1}{2} \times 14\tfrac{1}{4} \times \tfrac{1}{2} &= 131\cdot8125 \\
18\tfrac{1}{2} \times 3 \times 2 \times \tfrac{1}{2} &= 55\cdot5 \\
9 \times 3 \times 3 \times \tfrac{1}{2} &= 40\cdot5 \\
4\tfrac{1}{4} \times 3 \times \tfrac{1}{2} &= 6\cdot375 \\
\hline
& 234\cdot1875
\end{aligned}
$$

234·1875 cub. in. of cement mortar, assuming the bricks to be of true shape and without frog. This occupies $18\tfrac{1}{2} \times 14\tfrac{1}{4} \times 3\tfrac{1}{2} = 922\cdot6875$ cub. in. Then in 1 cub. yd. of brickwork there will be 27 × 1728 cub. in., containing $6 \times \frac{27 \times 1728}{922\cdot6875} = 303\cdot4$, say 304 bricks, and $\frac{27 \times 1728 \times 234\cdot1875}{922\cdot6875} =$ 11842 cub. in. or $\frac{11842}{1728} = 6\cdot853$ cub. ft. of cement mortar, without allowing any waste. Approximately, for a cub. yd. of mortar 25 bushels of sand and cement (3 to 1) are required, owing to shrinkage, namely, 6¼ bushels of cement, 18¾ bushels of sand, and 37 gal. of water; or for a cub. yd. of brickwork as above there will be required $6\tfrac{1}{4} \times \frac{6\cdot853}{27} = 1\cdot586$, say 1½ bushels, of Portland cement, and $1\tfrac{1}{2} \times 3 = 4\tfrac{1}{2}$ bushels of sand. The cost for the materials of a cub. yd. of brickwork will thus be :—

	s.	*d.*
Bricks, 305 @ 35s. per 1,000 ...	10	8¼
Sand, 4½ bushels @ 2½d.	0	11¼
Cement, 1½ bushels @ 3s. 6d. ...	5	3
	16	10½
Add profit, say 10 per cent.	1	8½
Cost of materials per cub. yd.	18	7

Take a second case. Assume that a bricklayer's wages are 1s. an hour, attendant's wages 6d. an hour, and that it is required to find the cost of plain brick walling 16 ft. to 30 ft. above ground. The attendant has to erect the scaffolding before the bricklayer begins, and he has to strike it at completion. One man will be mixing the mortar and carrying it, while a second is carrying bricks to the bricklayer. If a bricklayer lays 60 bricks per hour, it will take him $\frac{304}{60} = 5\cdot07$, say 5 hours, to build 1 cub. yd., and the cost will be :—

	s.	*d.*
Bricklayer, 5 hours at 1s. ...	5	0
Labourers, 10 hours at 6d. ...	5	0
Water, say	0	1½
Scaffolding and plant	0	6
	10	7½
Add profit, say 10 per cent.	1	0½
Cost per cub. yd. for labour ...	11	8

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MASONRY.

Building Stones.

The sandstones, limestones, granites, and slates are the principal building stones, and they will be discussed here in that order. The chief varieties of sandstones are Craigleith, Bramley Fall, and Forest of Dean ; and of limestones, Portland, Kentish Rag, Bath, and Caen, all these being given here in their order of durability.

Sandstones.

Sandstones as a class are hard and non-absorbent, but the individual quality depends upon the nature of the cementing material, the grains themselves being of quartz and practically indestructible. This cementing substance may be silica, carbonate of lime, carbonate of magnesia, alumina, oxide of iron, or mixtures of these. Carbonate of lime is the most liable to decay. A rough way of judging the durability of sandstone is by the appearance of the fracture—if this appears bright, clean, and sharp, the stone will probably weather well ; a dull, earthy appearance indicates liability to decay.

Yorkshire, or Hard York, and Bramley Fall Stone.—Yorkshire Stone, or Hard York, is a general term for sandstone from Yorkshire ; it is apt to flake in the flagstones from deficiency of cementing material between some of the layers. York Stone is obtained from the coal measures and millstone grit series. The colour is a light yellowish or ferruginous brown, but some varieties are bluish. Bramley Fall is the best quality of Yorkshire stone. It is coarse-grained, and very free from lamination. The original quarry is now worked out, but a somewhat similar stone is still sold under the same name. The original stone was largely used in engineering structures and for foundations of columns and machinery. The stone sold under this name is generally strong and durable, except when it contains an excess of grains of potash felspar, which makes it weather badly. Thin beds are generally inferior, and the stone is subject to iron stains.

Craigleith Stone.—This is the most durable sandstone in the United Kingdom. It is composed of quartz grains united by a siliceous cement, with small plates of mica. It contains 98 per cent. of silica, and only 1 per cent. of carbonate of lime.

Forest of Dean Stone.—"Notes in Building Construction" states that this stone is found in the Coal Measures near Lydney and Coleford in Gloucestershire. There are three distinct series or beds of considerable thickness ; the upper is a soft, easily worked stone of various degrees of hardness ; the second is harder, and the third harder still, and of a finer grit. Both the second and third series can be quarried in blocks of any size. The first and second series are of a grey colour, the third is bluer. Some of the stone has a brownish tint. The stone weathers well if placed on its natural bed. The expression "natural bed" is explained under that heading on p. 133. The stone is admirably adapted for building or for heavy engineering work such as bridges and docks.

Other Sandstones.—Other varieties of sandstone are Robin Hood, Park Spring, and Potter Newton. The finer grained sandstones are best for building, and the grits for engineering work. The flaking stones are generally used for pavings ; the easy cleavage is due to plates of mica. Sandstone is strong under pressure, but not suited for moulding or carving. Unless very flaky it all weathers well. Weight, 156 lb. per foot cube. Price, 3s. to 3s. 3d. per foot cube, delivered in London, or 6s. 6d. including all labour.

Testing Sandstones.—A durable sandstone should consist almost entirely of small uniform grains of quartz (silica or sand) cemented together with a siliceous cement. It should contain only a very small proportion of carbonate of lime, if any, and should be free from uncombined particles of iron. The colour may vary, but uniformity always accompanies the best qualities. Generally speaking, and with due regard to chemical composition, the harder and denser the stone is, the more durable it will be. The following tests may be applied:—A recent fracture examined under the microscope should be clean and bright, the more crystalline the better, and with the grains fairly uniform in size and well cemented together. A dull earthy appearance would betoken a liability to decay. C. H. Smith's test is to break off chippings about the size of a shilling, put them in a glass of clean water one-third full, and let them stand half an hour; then if the stone is good only a slight cloudiness of the water will be seen on agitating the glass.

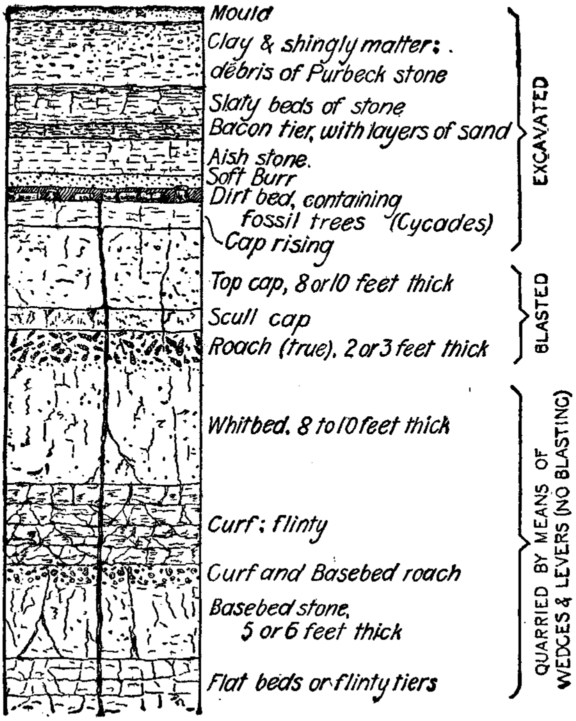

Mould
Clay & shingly matter; débris of Purbeck stone
Slaty beds of stone
Bacon tier, with layers of sand
Aish stone.
Soft Burr
Dirt bed, containing fossil trees (Cycades)
Cap rising
EXCAVATED
Top cap, 8 or 10 feet thick
Scull cap
Roach (true). 2 or 3 feet thick
BLASTED
Whitbed. 8 to 10 feet thick
Curf: flinty
Curf and Basebed roach
Basebed stone, 5 or 6 feet thick
Flat beds or flinty tiers
QUARRIED BY MEANS OF WEDGES & LEVERS (NO BLASTING)

Fig. 488.—Section of Portland Stone Quarry.

Brard's test is to weigh some small pieces when damp, and then immerse them in a concentrated boiling solution of sulphate of soda; upon removal the stone must be suspended a few days and re-weighed. The loss shows the probable effect of frost. Sometimes the stone is simply immersed in a cold solution. A chemical test to show the quantity of silica is delusive unless the stone is fairly uniform throughout; the durability depends very much upon the minute state of division of the silica and its uniform dissemination throughout the mass. If immersed in a weak solution of hydrochloric acid there should be but very slight effervescence, if any, indicating an escape of carbon dioxide from the carbonate of lime. Immersed in plain water for twenty-four hours the weight should not increase by absorption of water more than 5 per cent., and the weight in ordinary condition should not be less than 130 lb. per cub. ft. The best test is to visit buildings where the same stone has been used and see how it has weathered, making sure that it is from the same bed in the quarry, and that the atmospheric influences to which the building has been exposed are similar to those under which it is proposed to place the stone.

Limestones.

Portland Stone.—This is obtained from the Upper Oolite series of sedimentary rocks in the Isle of Portland, off the Dorsetshire coast. Fig. 488, adapted from an illustration in "Notes in Building Construction," gives some idea of the section of a Portland stone quarry. There are four varieties under this name, classified according to their position in the quarry, namely: True Roach, Whitbed, Bastard Roach, and Basebed. They are nearly alike in chemical analysis, consisting chiefly of carbonate of lime, an average composition being—

Silica	1·20
Carbonate of lime... ...	95·16
Carbonate of magnesia ...	1·20
Iron and alumina	0·50
Water and loss	1·94
Bitumen	Trace
	100·00

In the most durable stone the cementing material is semi-crystalline, and in the least durable it is in an earthy and powdery condition. The colour varies from a bluish-white to a light brown. The weight is 135 lb. to 152 lb. per cubic foot, or say 14½ cub. ft. to the ton. The price delivered in London is about 2s. 8d. per cubic foot, or sawn to scantling 3s. 6d., and including all labour, 7s. It has been used in the construction of St. Paul's Cathedral, Custom House, Goldsmiths' Hall, etc. Portland stone varies very much, according to the part of the quarry from which it is taken, and unless the particular bed is specified the result may be disappointing.

True Roach Portland Stone.—This is the strongest of the series, and consists of a mass of fossils united by a cement of carbonate of lime, and is distinguished from the Bastard Roach by the presence of the "Portland screw fossil" (Fig. 489). It is unsuitable for fine work owing to a number of cavities in and about the fossils, but it weathers well. It is admirably suited and largely used for engineering works of all kinds, being rough and strong, and resisting the action of water; thus it is particularly suitable for sea walls, breakwaters, etc.

Whitbed Portland Stone.—Whitbed, or whitebed, is perhaps the most valuable of the Portland series; it consists of fine oolite grains well cemented together by crystalline carbonate of lime interspersed occasionally with a small amount of shelly matter. It is a good weathering stone, easily worked, and capable of holding a sharp arris and a fine surface. The colour is white to whity-brown. It is used for window and door dressings, external carvings (when not too hard for the style required), fine ashlar work, and general construction.

Bastard Roach Portland Stone.—This resembles the True Roach in general appearance, but has not the Portland screw fossil. The cementing material is inferior, and the stone does not weather well. It may be used for covered interior work, but not for any external work.

Basebed Portland Stone.—This is so similar to the Whitbed that it cannot easily be distinguished except by its greater freedom from shelly material. Also, when examined through a magnifying glass, it is of a less roe-like texture than Whitbed. Being more uniform in texture and softer to work, it is preferred by masons, but does not weather so well. It is useful for internal work and carving, and is generally known as "best-bed." "Notes in Building Construction," comparing these stones, says, "Though these strata are so different in characteristics, the good stone can hardly be distinguished from the other, even by the most practised eye," and that the best way is "to have the stone selected . . . . in the quarry by some experienced and trustworthy man."

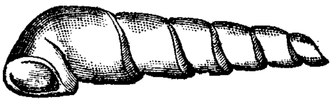


Fig. 489.—Portland Screw Fossil.

Kentish Rag Stone.—This is a compact limestone of a blue-grey tint, from the geological system known as Cretaceous and the formation known as Greensand. It is chiefly used for rubble walling and polygonal rag work, paving setts, curbs, and road metal. It is mixed with hassock, a soft calcareous sandstone, porous and perishable, which often adheres to it. There are many different beds or layers in the quarry of varying quality, those useful for ashlar work being nearer the bottom; but the stone is very difficult to dress, and often contains pockets of iron, which cause rust stains when exposed, so that it is not a high-class building stone, although good for many purposes. It is quarried by blasting, wedging, and the use of the crowbar. Pelsea yields large hard blocks 12 in. thick, difficult to quarry; Whiteland Bridge produces blocks 12 ft. long, any width, and 14 in. thick, stone very free working, bluish colour; Horse Bridge yields blocks of good stone 15 ft. long and 16 in. thick; other beds give smaller stone, suitable for curbing and pitching.

Bath Stone.—This is a typical oolitic granular limestone obtained from the Great Oolite or Bath series, all the principal quarries being in the neighbourhood of Bath. The colour varies from white to creamy yellow. It consists of grains of carbonate of lime cemented together by the same substance or by some mixture of carbonate of lime with silica or alumina. Sometimes it is interspersed with shelly fragments. When first quarried it is soft and moist, the moisture being called quarry sap, and if quarried in the winter is liable to disintegration by frost. It is generally very free-working, of a fine even grain, but is subject to sand cracks, vents, clay-balls, etc. Being very soft when first quarried, it is easily worked for fine carving, and fortunately it hardens subsequently on exposure to the air, so that it is a favourite stone for general use in ornamental window dressings in connection with brickwork. Brownish veiny marks may occasionally be seen on the finished surface of Bath stone; these are probably due to traces of iron in the composition. In good Bath stone there is not much difference in regard to strength in the various directions, as there is no lamination perceptible; at the most, only striæ or slight markings can be found on inspecting a finished surface. For outside dressings the stone should be well seasoned, hard, close grained, and capable of weathering well. Its weight is 123 lb. per foot cube. Its cost is about 2s. to 2s. 6d. per foot cube, delivered in London, or 4s. 6d. including all labour. It is obtainable in blocks of 10 to 12 tons each.

Varieties of Bath Stone.—Box Ground is the best weathering variety, although somewhat coarse, and is extensively used for dressings to brickwork in suburban London. Westwood Down and Lodge Stile Combe quarries are said to yield better stone. For internal carved work a softer stone of fine grain is desirable, and uniformity of texture is very important. For this purpose Farleigh Down is a typical kind, but Corsham Down, Stoke Ground, and Combe Down may also be used. The stone when first quarried is very soft, and the general contour of the carving should be roughed out or "boasted," leaving the fine work to be done after the stone is fixed in position. It is very difficult to determine whether newly dressed Bath stone has been fixed upon its natural bed, and the mason who works the stone is the best judge of its natural bed, as he can tell by the "feel" of the grain in working it.

Red Mansfield Stone.—This is a siliceous dolomite, or magnesian limestone containing a large proportion of silica or sand, found near Mansfield, Nottinghamshire. It is a good weathering stone in a clear atmosphere, suitable for ashlar work, columns, mouldings, carvings, etc.; but it loses some of its colour after exposure.

Caen Stone.—This is an oolitic limestone found in Normandy, of a pale cream-yellow colour, very soft when first quarried, but hardening on exposure. It is easily worked and carved, but weathers badly, and should be used only for internal work.

Durability of Limestones.—Limestones when compact, as marble, are tolerably free from defects other than those of colour. As a class they are rather soft and absorbent, and do not therefore weather so well as sandstones, particularly when they contain clay as an impurity. They are very liable to be acted upon by the acids in the air of manufacturing towns.

Granites and other Igneous Rocks.

Composition of Granite.—Granite varies much in composition, but if good it consists chiefly of about 50 to 60 per cent. quartz, 30 to 40 per cent. felspar, 10 per cent. mica. Other good proportions are:—About 40 per cent. quartz, 50 per cent. felspar, and 10 per cent. mica. The quartz consists of silica in hard glassy amorphous or crystalline lumps, whitish grey or colourless. The felspar consists of lime and soda in lustrous crystalline masses of various sizes, and of different colours in samples from the different quarries, such as white, grey, pink, or red. It is this substance that gives the general colour to granite. The mica consists of semi-transparent glistening scales, generally dark grey or black, which can be flaked with a knife. There are also traces of iron. Granite with a fine grain is suitable for polished columns and other ornamental work; that with larger grains of the various constituents may be used for steps, plinths, etc., where hardness and durability are required.

Weathering Properties of Granite. — The weathering properties of granite depend upon the proportion and quality of the felspar and mica, quartz, the remaining constituent, being exceedingly durable. Potash felspar (orthoclase) and a lime and soda felspar (oligoclase) are those most commonly found in granite, the potash felspar being the more liable to decay. The felspar should not contain an excess of lime, iron, or soda. Mica is always readily decomposed, but especially when containing an excess of lime, iron, or soda. The quantity of iron in the felspar or mica affects the durability whether occurring as the oxide or in combination with sulphur. The white pyrites (marcasite) sometimes present also decomposes quickly. The smaller the grains of which the granite is composed, the more evenly will it wear. The durability of granite can be judged in a rough way by the colour and quantity of the felspar and mica. The felspar should appear crystalline and lustrous, not earthy. The hardness of the felspar and mica may be tested by the point of a penknife. If the stone is discoloured in patches with dark yellow stains, it should be rejected because of the iron in its composition.

Varieties of Granite. — Two well-known varieties are (*a*) Aberdeen, durable and of good appearance, and (*b*) Guernsey, a good weathering stone, very hard and durable, used for paving work, but apt to become slippery. English granites come chiefly from Devon and Cornwall; Shap granite is obtained in Westmoreland; of Irish granites, the Wicklow variety is most used.

Strength of Granite.—Granite has a maximum crushing strength of seven to nine tons per square *inch*, and may be used under a working load of fifteen to twenty-five tons per square *foot*. The extraordinary precautions taken in testing stone, to produce a high result, render the figures of crushing strength very deceptive. The contingencies in the actual use of building stones make a nominal factor of safety of 50 none too much.

Flaws in Granite.—The felspar grains, particularly if potash felspar or orthoclase, may be in a state of decay causing disintegration, or if sound but large may render the stone weak; they occur mostly in Devonshire and Cornish granites. Irregular dissemination of component parts is also a flaw. Patches and dark stains of iron if present will oxidise, and light patches or streaks of marcasite or white pyrites would rapidly decompose. Mica and talc in unusual quantity are serious defects, as they decay more rapidly than any of the ordinary constituents.

Hornblende.—Hornblende, diallage, and talc all occur in the igneous rock formations, and their durability is in this order. Hornblende is a dark green or blackish mineral in prismatic crystals consisting of silica, magnesia, and lime. It is the typical constituent of syenite or greenstone, distinguishing it from and rendering it more durable than granite, and taking the place of the mica of granite, but in a syenitic granite both minerals occur. Hornblende is more durable than mica, and is distinguished from it by being brittle instead of elastic.

Diallage.—This is one of the constituents of serpentine, or soapstone, and of some varieties of syenite. It is a pale or green variety of hornblende and dolomite, having good cleavage, and may be described as a foliated silicate of magnesia. It does not weather so well as hornblende.

Talc.—This is a soft whitish mineral with a greasy feel, and scales off in thin flakes. It consists chiefly of magnesia and silica, being a form of mica, and is found in talcose granite in addition to the ordinary ingredients. It does not weather well.

Serpentine.—This belongs geologically to the igneous formation with basalt, syenite, etc. The mottled appearance is supposed to resemble the skin of a serpent, hence the name. It is found principally in Cornwall; also in Anglesea, Banffshire, Aberdeenshire, Shetland Isles, Galway in Ireland (Connemara marble), Donegal and Sligo counties. Pure serpentine is a hydrated silicate of magnesia, but is generally found intermixed with carbonate of lime, steatite or soapstone, and diallage (a foliated green variety of hornblende and dolomite). It is found in all shades of green and red with mottled streaks and patches. Serpentine is compact in texture, not brittle, is easily worked, so soft that it may be cut with a knife, and is capable of receiving a fine polish. It is generally quarried in blocks 2 ft. to 3 ft. long. Its chief use is for indoor decorative work, as it does not weather well, especially in the neighbourhood of towns; as a rule the red varieties weather better than the green. It is made into table tops, shafts of columns, pilasters, chimney pieces, and ornaments.

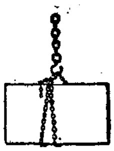


Fig. 490.—Stone in Sling Chain.

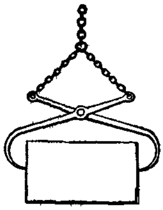


Fig. 491.—Stone in Lifting Tongs.

Basalt.—This belongs to the same geological formation as Serpentine, in the division known as "trap" rocks. It comes from Rowley Regis in Staffordshire, where it is known as Rowley Rag, and the counties of Armagh, Antrim, and Londonderry; it is also found at Portrush. Basalt is composed of lime felspar, augite (silicate of magnesia), olivine, and titano-ferrite. The colour varies from greyish to black; in the lighter coloured, felspar predominates; in the darker, iron or a ferruginous augite. It is often of a dark green, similar in appearance to whinstone. It occurs in dykes or sheets penetrating or lying between older rocks, or upon the surface, sometimes stratified and at other times columnar with six or eight sides, as in the Isle of Staffa or Giant's Causeway. Owing to its hardness and resistance to crushing, it is eminently adapted for paving setts, curbs, etc., and for making artificial stone on Chance's system.

Syenite.—This is an igneous rock similar in appearance to granite, taking its name from Syene in Upper Egypt. Its localities are Leicestershire, Merionethshire, and the Channel Islands. True syenite consists of crystals of quartz, felspar, and hornblende, the latter taking the place of the mica in ordinary granite. When both hornblende and mica are present it is a syenitic granite. The colour is generally dark green, owing to the hornblende; it contains some pink, grey, or pinkish brown crystals of felspar, according to locality quarried. Syenite is on the whole tougher, harder, more compact, and of a finer grain than ordinary granite, and generally of a darker colour. It is very durable, and is the best of the granite class. It is used principally for paving setts and road metal, although it will take a fine polish, and is suited to external ornamental purposes when the expense is justifiable.

Slates.

The chief varieties of slate are—Welsh, with a good cleavage, splitting very thin, and Westmoreland, hard, tough, and very durable. The weathering properties of slates depend on their non-porosity, and the absence of white iron pyrites (marcasite).

Tests for Slates.—A good slate should give out a sharp metallic ring when struck with the knuckles; it should not splinter under the slater's axe, it should be easily "holed" without fracture, and should not be tender or friable at the edges. The following rough tests may also be used: (1) Weigh the slate carefully when dry, steep it in water for twenty-four hours, run the water off, and weigh again; any difference of weight will show the amount of absorption. (2) Stand the slate in water up to half its height—if it be of bad quality the water will rise in the upper half, but in a good slate no sign of moisture will be seen above the water-line. (3) Breathe on the slate. If a clayey odour be strongly emitted, it may be inferred that the slate will not weather. The subject of roofing slates is fully treated in a section on roof coverings given later in this work.

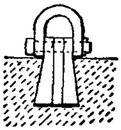


Fig. 492.—Lewis.

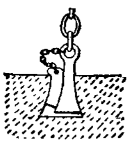


Fig. 493.—Lewis with Plug on Chain.

Stones for External Work in London.

The best stones for external work in London are the following, in the order given:—Granite, Craigleith, Portland (Whitbed), Mansfield, Doulting, Bath (Box Ground). The atmosphere of London contains various acids carried chiefly by the smoke, such as sulphuric acid and nitric acid, which act upon the carbonate of lime which forms the basis or matrix of many varieties of building stone, causing a gradual disintegration. Added to this, the alternate moisture and frost would crumble any stone which was sensibly porous; and, in view of these considerations, Craigleith is the most durable stone in London, consisting of 98 per cent. of silica, and having a good cementing material or matrix.

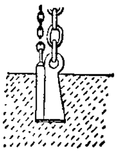


Fig. 494.—Lewis for Work under Water.

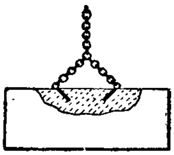


Fig. 495.—Lifting-pins in Granite.

Natural Bed of Stone.

The natural beds of building stones are the planes of stratification, generally having a reduced cohesion, which have been formed by the successive layers of deposit through water, and generally occur more or less horizontal in the quarry. Owing to their manner of formation, granites, syenites, and other igneous and crystalline rocks, do not show any bedding, and some limestones show very little. Cleavage applies more particularly to slates, and means the facility with which the slates may be separated in planes distinct from the original sedimentary stratification or bedding. It has been suggested that the aggregations of the atoms of the slates were originally circular, that the weight of superincumbent deposits compressed these aggregations into horizontal directions, and that the contraction of the earth's crust in cooling caused the aggregations to take a more or less vertical position by pressure, and so formed the cleavage planes. Returning to building stones proper, the planes of bedding should be vertical and at right angles to the face in the deeply undercut cornices, to prevent the possibility of a piece dropping off by the separation of two layers. The keystones of arches should also be built with the planes of bedding vertical, in order to resist most effectively the pressure upon them from the thrust set up in the arch, but in other cases the natural bed should be horizontal.

Strength and Weight of Stone.

According to Rennie, the crushing weight per square inch is, for blue Aberdeen granite 4·87 tons, Craigleith sandstone 2·45 tons, and Portland limestone 2·03 tons; but the strength of stone differs considerably, and according to other authorities some varieties of each of these classes of stone are as low as 1 ton per square inch. The weight of stone also varies, granite being from 162 lb. to 187 lb. per cubic foot, sandstone 116 lb. to 170 lb., and compact limestone 155 lb. to 172 lb. Blue Aberdeen granite weighs 167 lb., Craigleith 145 lb., Portland basebed 145 lb. The safe working load in ordinary cases would be—granite 15 tons per foot super., Portland and compact limestone 15 tons, hard York stone 12 tons, with a maximum increase of 50 per cent. for dead loads under favourable circumstances. The working load on a sandstone pier in one block, well bedded, is 12 tons per foot superficial for a live load, or 18 tons maximum for a dead load. On a built-up stone pier of cube stone set in cement not exceeding six diameters high, anything up to one third the above would be safe, according to the workmanship of the pier. If of rubble stone, the strength of pier would be at least halved again.

Gripping Stone Blocks for Lifting.

Fig. 490 shows the common method of lifting any load by a sling chain. Blocks of stone are apt to be chipped by this method unless strips of wood are inserted to keep the chain off the edges. Fig. 491 shows lifting tongs, commonly used for raising Bath or other soft stone blocks; the heavier the load the tighter the grip naturally produced. Fig. 492 shows the common lewis in a dovetailed mortice for lifting hard stone. This takes apart into several pieces. The plug consists of three parts, and is fixed by first inserting the taper pieces, then the parallel piece, which pushes the others into the dovetailed recesses; the

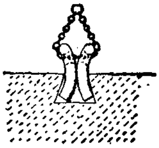


Fig. 496.—Lifting-pins in Lewis Mortice.

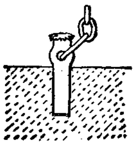


Fig. 497.—Taper Plug in Granite.

shackle is then placed over the top, the pin put through, and the cotter or split pin in the end. It is removed in the reverse order for insertion in the next stone. Work finished before hoisting is often lifted in this way, as the ends and sides are not liable to damage. Fig. 493 shows a somewhat similar arrangement, but without any loose parts. The mortice is curved and cut deeper than the plug, so that when the plug is down at the bottom the key can be inserted, while the lifting of the plug puts it all tight. Fig. 494 shows the application to work under water, where, by a separate chain to the surface of the water, the key can be withdrawn after the stone is lowered into place. This figure shows a better form for the plug and key than Fig. 493, and equally suitable if the small chain be left long enough to withdraw the key. Fig. 495 shows the arrangement of lifting-pins for granite. They are inserted loosely into inclined mortices, and hold the stone securely when the lifting chain is tightened. Fig. 496 is similar in principle, but arranged differently. Fig. 497 consists of a plug with a slight taper driven firmly into a parallel hole in granite, being loosened by a few side blows after the stone is set. This is not safe for use singly, and is better when two are used, as the bridle chains then pull the tops together instead of pulling straight from the mortice. Fig. 498 shows a block of rock-faced ashlar in course of lifting, or lowering to its bed ; two hocks (dogs) and a chain serve to make a hold for the hook of the hoisting machine; the rough back and face prevent slipping.

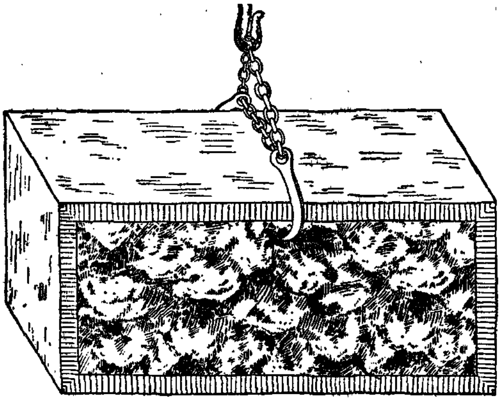


Fig. 498.—Rock-faced Ashlar lifted with Dogs.

Tools for Dressing Stone.

Some of the tools used in quarrying and dressing the stone are shown in Figs. 499 to 512, taken chiefly from Seddon's "Builders' Work." Pickaxes and shovels are also used. The tools in more particular use for dressing granite are shown by Figs. 513 to 517. The mash hammer (Fig. 510) is also used.

Facework on Masonry.

Some idea as to the modes of faceworking masonry can be gained from Figs. 518 to 532, which show all the styles in general employment, except the rubbed or polished, for which an illustration is not necessary.

Rusticated Work.—Rusticated is the name given to the face of a stone when it is made to represent a rough, rock-like surface (see Fig. 498). It is known as rock-faced, rustic work, quarry-pitched, hammer-faced, hammer-blocked, hammer-dressed, etc. The edges next the joints are made to a true line, either flat or chamfered, in order that stones may be set truly in the courses. Rusticated is also a term for columns having the shaft composed apparently of alternate square and cylindrical blocks; and is also used for ashlar work having projecting flat faces with deeply recessed margins.

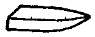


Fig. 499.—Gab.

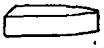


Fig. 500.—Wedge.

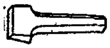


Fig. 501.—Pitching Tool.

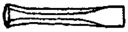


Fig. 502.—Inch Tool.

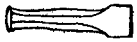


Fig. 503.—Boaster.

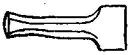


Fig. 504.—Broad Tool.

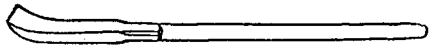


Fig. 505.—Crow Bar.

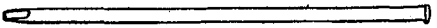


Fig. 506.—Crow Bar.

Droved or Boasted Work. — Droved work (Fig. 527) is a Scotch term for boasted work, which is a variety of chiselling on stone in which the marks of the tool run in parallel lines, each successive stroke being made beneath the last, down the whole length of the stone. The same operation is repeated until the marks extend over its whole breadth, although they do not run in continuous lines as in tooled work (Fig. 528).

Dressing Granite.—Col. Seddon, to whom most of the following information on dressing granite is due, says that granite is dressed by means of heavy picks and axes, after having been roughly shaped with the scabbling hammer. Mouldings, rebates, etc., are cut by means of iron chisels, steeled at the cutting edges, and used with a small hand hammer, called a mash hammer (Fig. 510). Granite, grit, and other hard stones, built into walls with their faces merely scabbled, are said to be quarry-pitched, hammer-faced, or hammer-blocked. Such work is called rock or rustic work, and is mostly confined to foundations, plinths, and quoins, where a bold, massive appearance is aimed at.

Hammer-faced. — Hammer - faced, hammer-dressed, or hammer-blocked work is done with the scabbling or spalling hammer. Thus squared stones for the quoins or face of a wall, merely left rough from the hammer, would be termed hammer-faced ashlars; the term ashlar, in such a case, is taken to mean square blocks 12 in. deep on face, and upwards, squared stones under 12 in. deep being called shoddies.

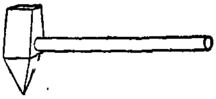


Fig. 507.—Scabbling Hammer.

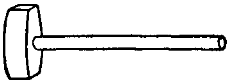


Fig. 508.—Spalling Hammer.

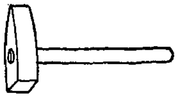


Fig. 509.—Waller's Hammer.

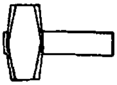


Fig. 510.—Mash Hammer.

Fig. 511.—Mallet.

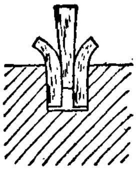


Fig. 512.—Plug and Feathers.

Scabbled.—Scabbled or roughly picked work is done with a pick, such as in Fig. 513, sometimes called a scabbling pick, and weighing about 20 lb., which takes down the excessive irregularities on hammer-faced work.

Punched.—Punched or puncheoned work is wrought to a finer face with a blunt pick (Fig. 514) called a punch or puncheon.

Picked.—Picked work is brought to a finer face with the pick shown in Fig. 513. Close or finely picked, dabbed or daubed work is done with a fine-pointed pick, or with a serrated pick, as in Fig. 515, leaving a surface as smooth as the process will admit of.

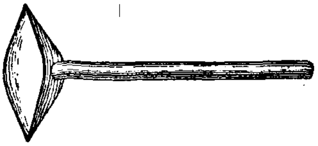


Fig. 513.—Scabbling Pick.

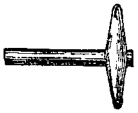


Fig. 514.—Punch or Puncheon.

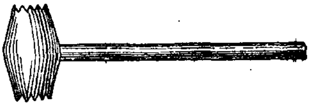


Fig. 515.—Serrated Pick.

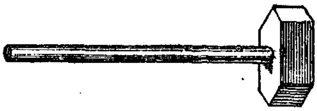


Fig. 516.—Axe.

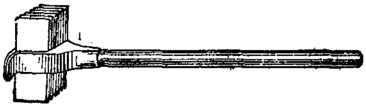


Fig. 517.—Patent Axe.

Draughted Margins.—It is usual to run a draught, or smooth surface, an inch or more in breadth, round the margins of squared stones, even when dressed only with the hammer or pick, in order to insure close-fitting joints. The stones are then said to be hammer-faced, or, as the case may be, with draughted margins. These margins are wrought with the axe as in single and fine axing.

Single Axed.—In single axed work the inequalities left by the pick are reduced by an axe weighing about 9 lb. (Fig. 516). Axed work shows the mark of the tool in parallel lines, and is used in quoins, rebates, cornices, etc. Fine axed is a more carefully wrought description of single-axed work.

Patent Axed.—Patent axed is the finest description of surface work before polishing, and is produced with a hammer or axe the faces of which are formed of a number of parallel thin steel blades bound together, so as to allow of their being taken out and re-sharpened (Fig. 517).

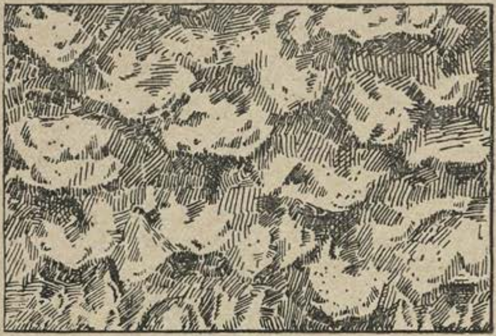


Fig. 518.—Rock-faced Work.

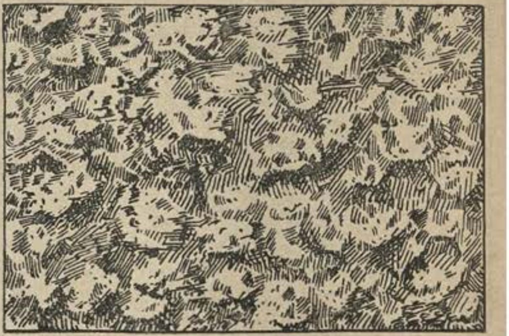


Fig. 519.—Hammer-dressed Work.

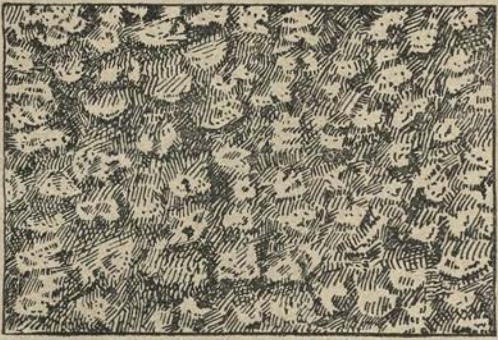


Fig. 520.—Scabbled Work.

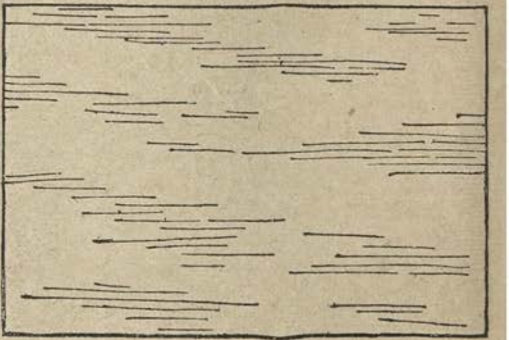


Fig. 521.—Sawn Work.

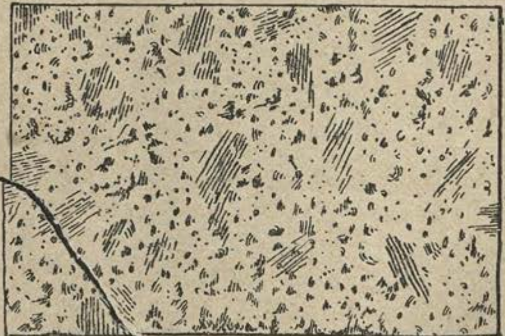


Fig. 522.—Half-plain Work.

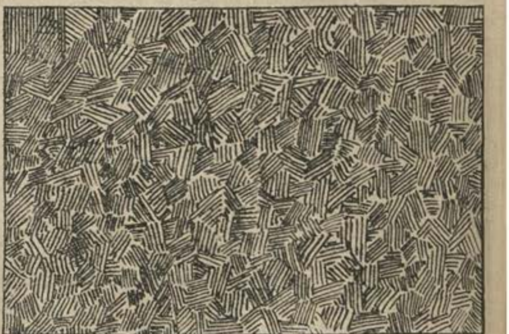


Fig. 523.—Plain Work.

Polished.—Polished work is performed by rubbing, first with fine sand and water, under an iron rubber, then with emery, and, lastly, with putty and flannel. Of late years the sand has been displaced by iron or steel filings, called shot, which is much more efficient than sand. All plain surfaces and running mouldings can be done by machinery but carvings and broken surfaces have to be done by the hand. Hard stones, such as granite, show off to best advantage when polished; but if such a high finish is considered too costly, it is better not to waste money on too fine a face, which only destroys the beauty of the grain and produces a flat, monotonous surface.

Aberdeen Practice.—The punch or puncheon used in Aberdeen is shaped as in Fig. 533, and is either oval or octagonal in section. With the puncheons and chisels there is used a hand hammer, termed a maul, which weighs from 3 lb. to $4\frac{1}{2}$ lb., and has a flexible hickory handle to give it spring on the hard stone. The kind of hammer-face or rustic-face that is mostly used in Aberdeen is worked thus:—

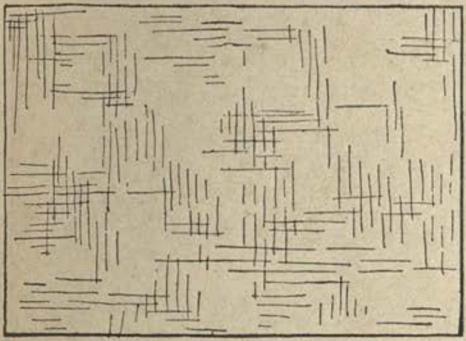


Fig. 524.—Dragged or Combed Work.

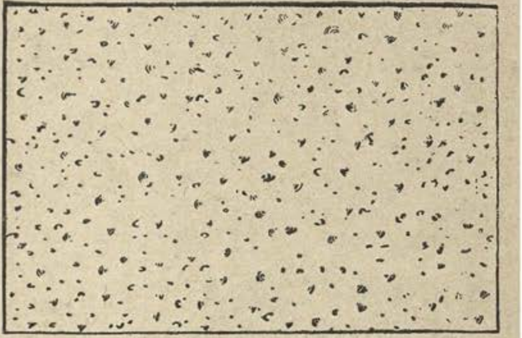


Fig. 525.—Sparrow-picked Work.

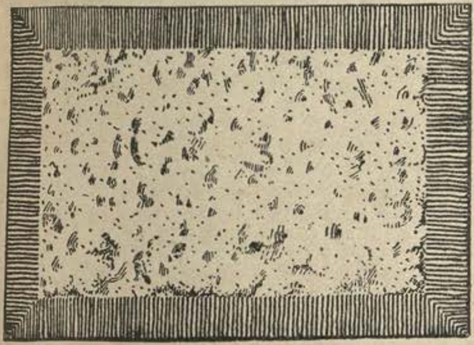


Fig. 526.—Pointed, Dabbed, or Punched Work with Drafted Margin.

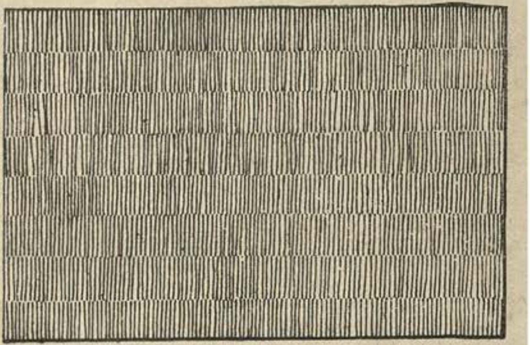


Fig. 527.—Boasted or Droved Work.

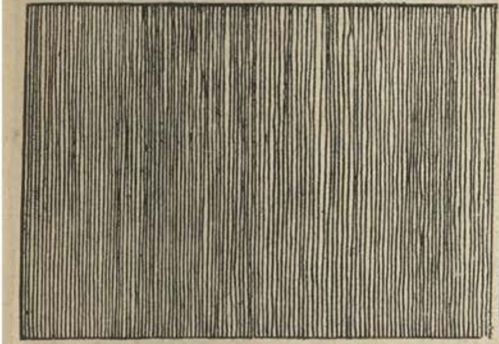


Fig. 528.—Tooled Work.

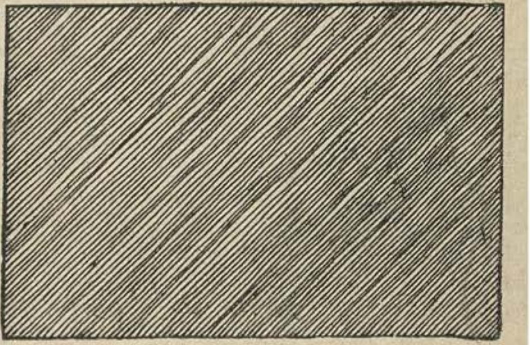


Fig. 529.—Stroked or Striped Work.

Run a draught round the stone on beds and joints next to face with a chisel ; puncheon off superfluous stone from beds and joints, and pitch off (the face kept out of wind) from this draught, and the stone is finished ; the face is not touched. The blocks thus treated are mostly used as quoins, as bases, and for ashlar work. In puncheoned ashlar, the bed, joints, and face are all puncheoned ; the face, of course, receiving most attention.

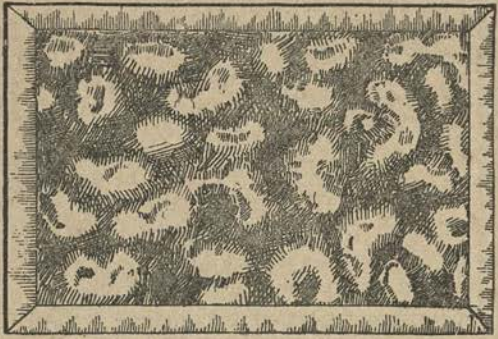


Fig. 530.—Rusticated and Chamfered Work.

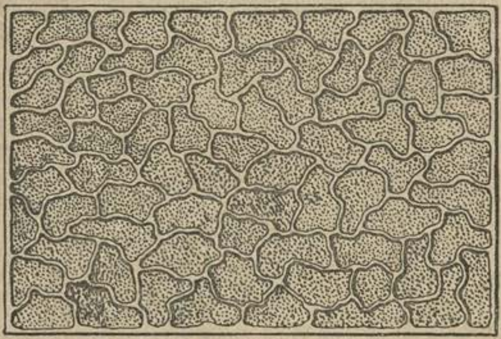


Fig. 531.—Reticulated Work.

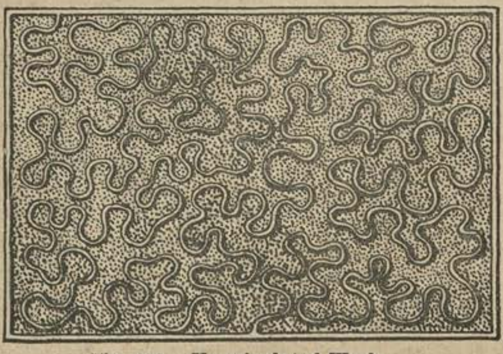


Fig. 532.—Vermiculated Work.

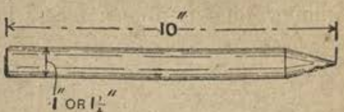


Fig. 533.—Puncheon for Working Granite.

Dressing True Face on Rough Block.

Assume that the face A B C D (Fig. 534) is to be dressed to a true plane. Dobson describes the method of doing this as follows : The mason first knocks off the superfluous stone (beginning from the lowest corner) along one edge of the block, as C D (Fig. 535), until it coincides with a straightedge throughout its whole length ; this is called a chisel draught. Another chisel draught is then made along one of the adjacent edges, as D A, and the ends of

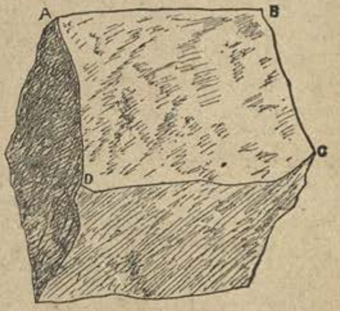


Fig. 534.—Rough Block to be Dressed.

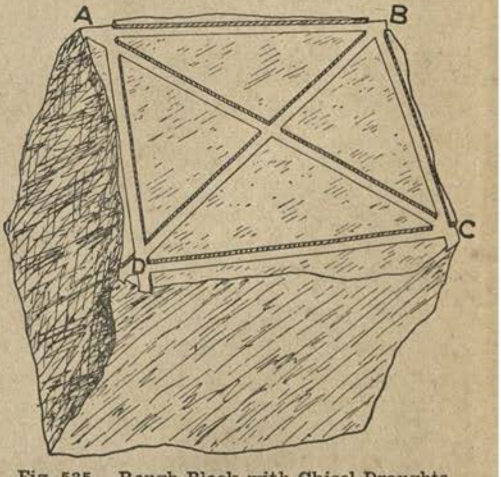


Fig. 535.—Rough Block with Chisel Draughts.

the two are connected by another draught, as C A; a fourth draught is then sunk across the last, as D B, which gives another angle point B, in the same plane with C, D, and A, by which the draughts B C and B A can be formed; and the stone is then knocked off between the outside draughts until a straightedge coincides with its surface in every part. But some consider that it is better to work a chisel draught on opposite edges of the stone first, to get the face out of winding. A more modern method of forming a true face to a large block of stone, as described by a correspondent in *Building World*, is as follows: Begin by chiselling at each corner of the block a sinking sufficiently low to take out any irregularities, then use four small cubes of beech or other hard wood about 2 in. square and exactly similar, called "boning pegs," placing one cube in each sinking. Across the top of two adjacent cubes or pegs lay a straightedge, and across the other two a similar straightedge. Then by standing at right angles to, and with the eye in line with, underside of one straightedge, sight or "bone" through to the other, and if not true, adjust the sinkings until the bottoms of the straightedges are out of winding. This having been done, work straight draughts from one sinking to another along opposite edges, until there is a draught all round the outside.

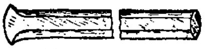


Fig. 537.

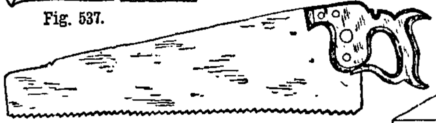


Fig. 536.

Fig. 536.—Handsaw. Fig. 537.—Jumper.

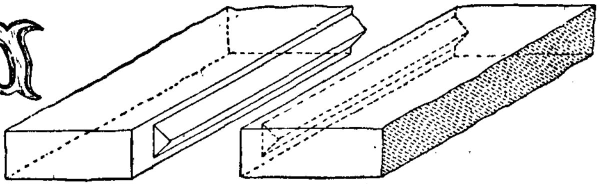


Fig. 540.—Joggle Joint in Landing.

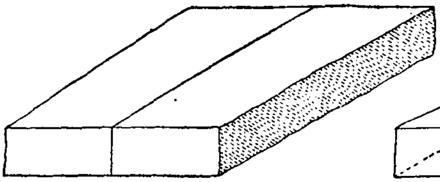


Fig. 538.—Butt Joint.

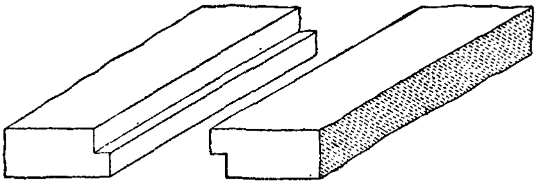


Fig. 539.—Rebated Joint.

Then point off superfluous stone and dress to a finished face in parallel draughts.

Dividing Stone Blocks.

Bath stone being soft, particularly when fresh from the quarry, can be cut with an ordinary handsaw (Fig. 536). Granite, being a very hard stone, is separated by drilling holes in a row along the line of separation, by means of a hammer and drill, or by means of a jumper (Fig. 537), and then inserting gabs, or wedges, or split plugs and feathers (see Figs. 499, 500 and 512), and driving them down uniformly.

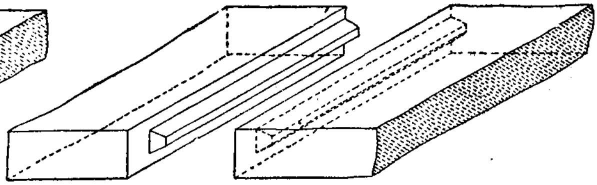


Fig. 541.—Joggle Joint in Landing.

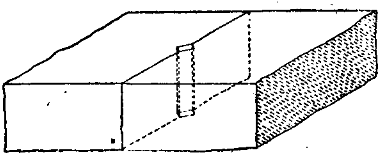


Fig. 542.—Double Arris Joggle with Pebbles in Cement.

Joints in Masonry.

The simplest joint is the plain butt joint (Fig. 538), the rebated joint (Fig. 539) coming next. Joggle joints are of two or three principal kinds (see Figs. 540 to 542); and it may be said that, in general, a joggle is a projection

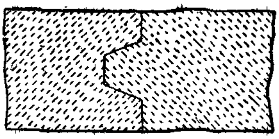


Fig. 543.—Section of Joggle Joint in Stone Landing.

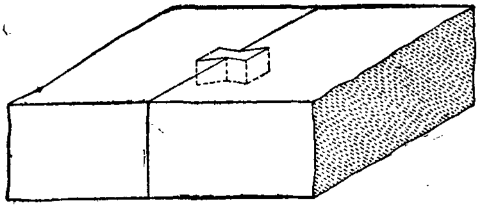


Fig. 544.—Dovetail Key or Cramp Joint.

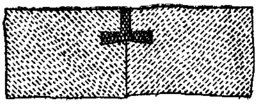


Fig. 545.

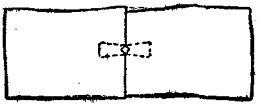


Fig. 546.

Figs. 545 and 546.—Lead Plug in Coping.

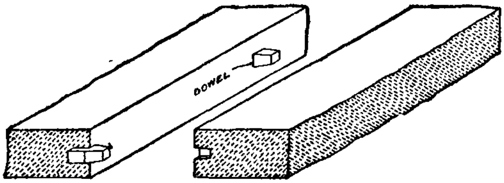


Fig. 547.—Dowelled Joint.

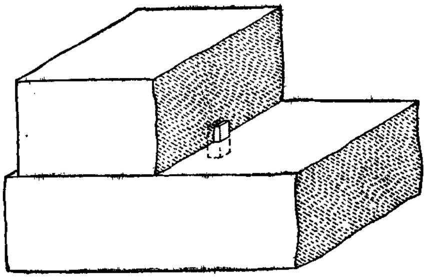


Fig 548.—Bed Plug in Ashlar Work.

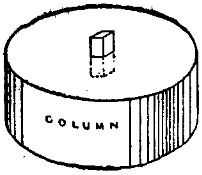


Fig. 549.—Bed Dowel.

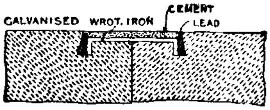

CALVANISED WROT. IRON
CEMENT
LEAD

Fig. 550.

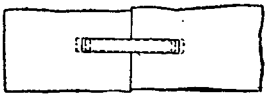


Fig. 551.

Figs. 550 and 551.—Cramp in Coping.

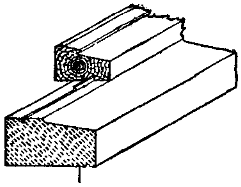


Fig. 552.—Water Bar Joint in Window Sill.

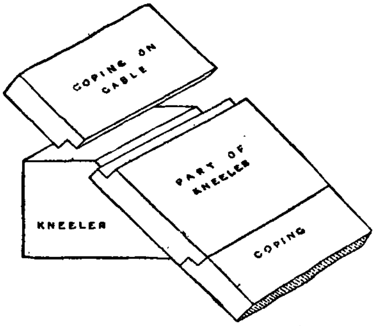

COPING ON GABLE
PART OF KNEELER
KNEELER
COPING

Fig. 554.—Kneeler and Coping.

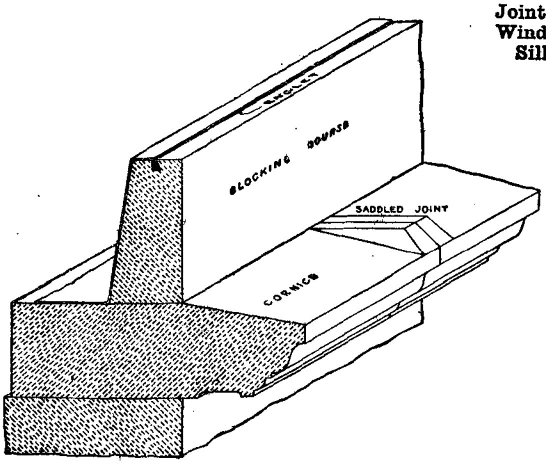

BLOCKING COURSE
SADDLED JOINT
CORNICE

Fig. 553.—Saddled Joint in Cornice, etc.

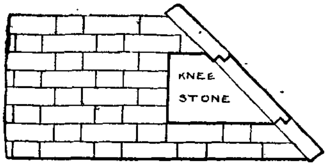


Fig. 555.—Kneeler or Kneestone in Gable Wall.

to prevent movement, such as on the bottom of a cast-iron stanchion, or on a roof-shoe; it is also an angular projection in the joint of a stone landing, as already shown in Figs. 540 and 541, and now illustrated in enlarged section by Fig. 543. It is also the term used for a bend in a parallel plane of an angle iron or T-iron in girder work and roof work. Dowels or cramps of slate, lead, or cast-iron are used as in Fig. 544, so as to form dovetail keys. Lead plugs or dowels formed by pouring molten lead into prepared cavities from the surface of the joint, as in Figs. 545 and 546, are commonly used for coping stones on dwarf walls; horizontal dowels of slate, as in Fig. 547, are used for the same and other purposes; and vertical slate dowels, or bed plugs, as in Figs. 548 and 549, are used in the joints of the shafts of stone columns and pilasters. But joints can also be secured with galvanised wrought-iron cramps, lead being poured in so as to make a dovetail key (see Figs. 550 and 551), the cramp being covered with cement. A water bar joint in a window-sill is shown in Fig. 552. A saddled joint in a cornice, together with a blocking course, into which lead flashing is secured with a raglet, is shown by Fig. 553. The method of jointing a gable coping where it rests on the kneeler or kneestone is illustrated in Fig. 554. Fig. 555 shows part of a gable wall in red brickwork, Flemish bond, with stone coping and kneestone to prevent the coping from sliding. In Fig. 556 the apex stone A is the topmost piece of coping, and kneelers are placed as at K K.

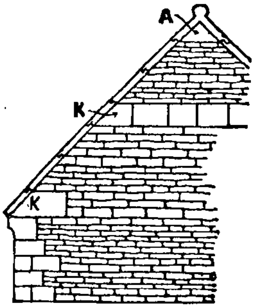


Fig. 556.—Coping and Kneelers.

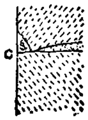


Fig. 560.—Flush Joint.

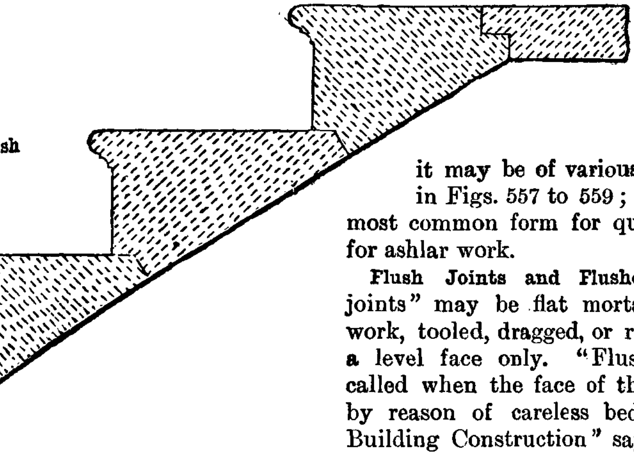
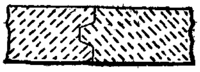


Fig. 561.—Section of Stone Steps and Landing.



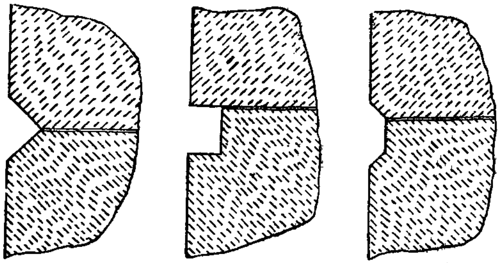


Fig. 557. Fig. 558. Fig. 559.

Figs. 557 to 559.—Rusticated Joints in Masonry.

Rusticated Joints.—When the face of stonework is rubbed and rustic jointing is required, it may be of various sections, as shown in Figs. 557 to 559; but Fig. 557 is the most common form for quoins, and Fig. 558 for ashlar work.

Flush Joints and Flushed Joints.—"Flush joints" may be flat mortar joints in ashlar work, tooled, dragged, or rubbed, that is, with a level face only. "Flushed joints" are so called when the face of the stone is chipped by reason of careless bedding. "Notes in Building Construction" says: "When, owing to the bed joints being worked hollow, the entire weight is thrown on the point C (Fig. 560) causing a 'spall or pieces, to be splintered off, the joint is said to be 'flushed.'" Seddon, in "Builders' Work," says: "Care should be taken to prevent the use of flush joints, which are formed by hollowing the beds below the plane of the chisel draughts round the edges. This was sometimes done by the Greeks in order to get perfectly close joints; but, by throwing all the pressure on the edges of the stones, they frequently splinter off and spoil the look of the work. As flush joints cannot be detected after the stones are laid, the masons must be looked after while at work upon them." From the above it will be clear that the terms "flush" and "flushed" are used indiscriminately.

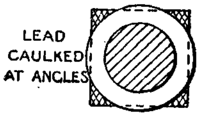


Fig. 562.

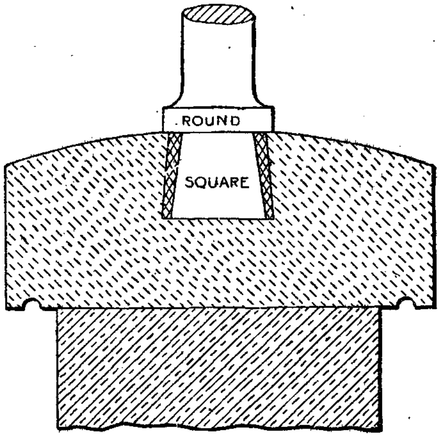


Fig. 563.

Figs. 562 and 563.—Jointing Iron Standard to Saddle-back Coping.

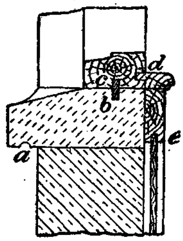


Fig. 564.—Five Kinds of Grooves.

Joints in Stone Stair and Landing.—Fig. 561 represents a section through a landing and some stone steps; the joints of treads to risers are simple, and are clearly shown; the joint in the landing is joggled, as in Fig. 543.

Cast-Iron Standard on Saddle-back Coping.

Fig. 563 shows a cross section through a saddle-back stone coping 12 in. by 5 in., throated and carrying an iron railing. It also shows the foot of a cast-iron standard 2 in. diameter securely leaded to the coping. A plan of the railing is shown by Fig. 562.

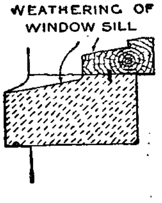


Fig. 565.

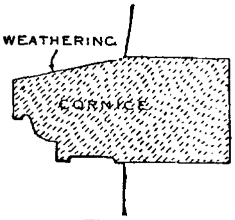


Fig. 566.

Figs. 565 and 566.—Weathering of Window Sill and Cornice.

Grooves in Masonry.

A groove is a narrow channel cut out in stone or wood to throw off moisture or to enable a connection to be made with another piece, as in Fig. 564, where *a* is a throating groove in a stone window sill to throw off moisture; *b*, groove in same for metal tongue; *c*, groove in oak sill for metal tongue; *d*, groove in same for window board; *e*, groove in top rail of window back for panel.

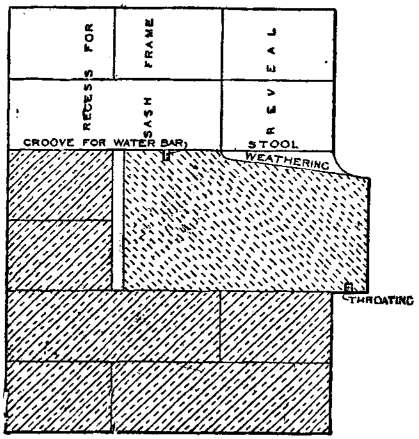


Fig. 567.—Section of Window Sill fixed in 14-in. Wall.

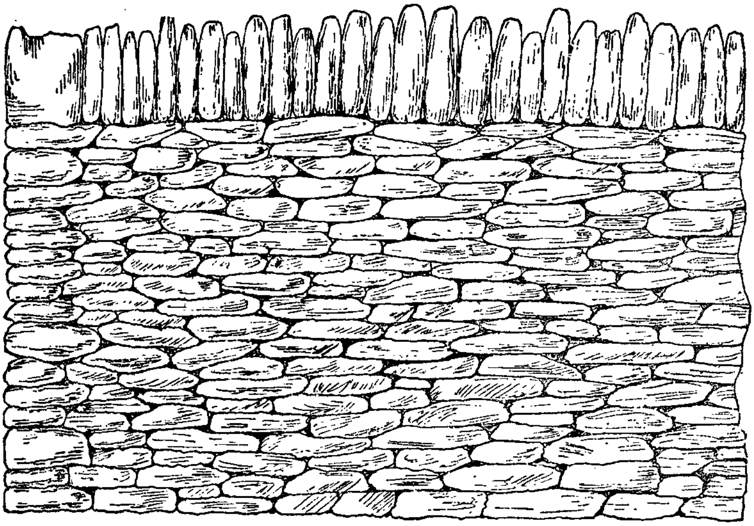


Fig. 568.—Flat-bedded Rubble laid dry with Rubble-on-edge Fencing.

Weathering.

Weathering (verb) a window sill is dressing the upper surface to a slope; or (noun) it is the sloped upper surface, to throw off rain (see Fig. 565). Weathering of stone may be taken as on the stone sill above, or on a stone cornice (see Fig. 566), but weathering is a term also used to refer to one of the properties of stone, meaning its durability under exposure to the weather.

Window Sill.

Fig. 567 represents the finished section of a window sill, weathered, throated and grooved, and fixed in a 14-in. brick wall. Two courses of brickwork are shown below the sill, and two courses in elevation above the stool.

Building Masonry Column.

The interposing of sheet lead between the stones of a column is not advisable, as the lead, by its lateral expansion under intense pressure, is liable to chip the edges off the stones. "With a view of guarding against the splintering or spalling of the arisses of cut stonework, as in columns carrying heavy weights, 7 lb. or 8 lb. sheet lead is frequently interposed between the stones. The lead, which is not allowed to

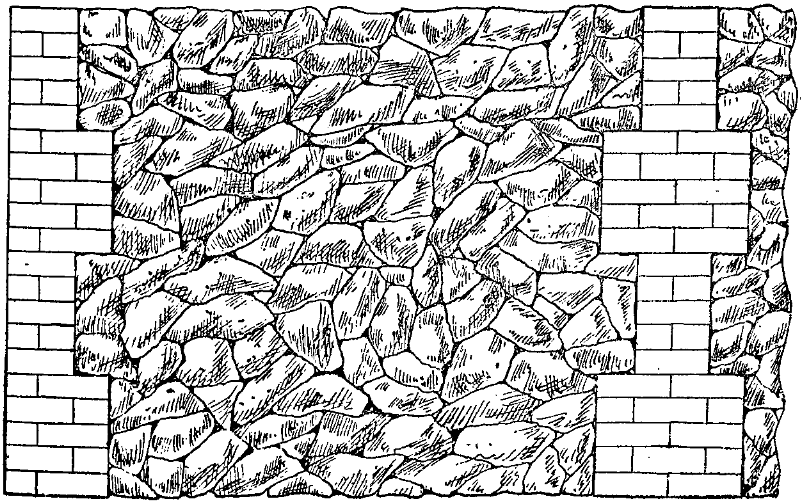


Fig. 569.—Uncoursed or Random Rubble with Brick Quoin and Pier.

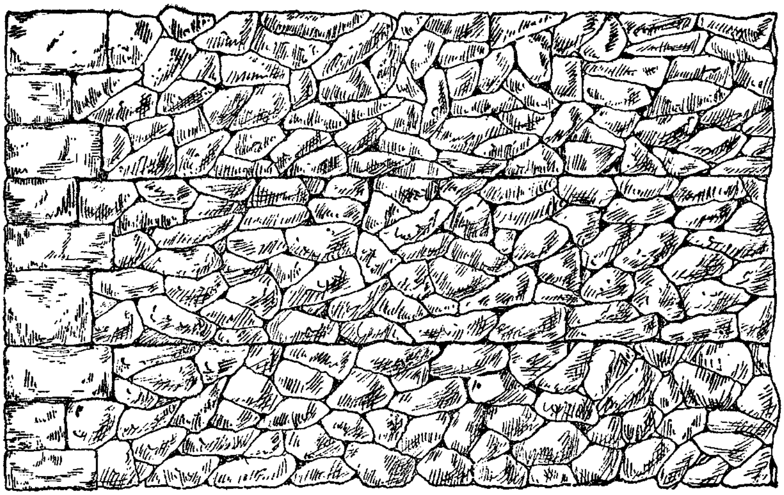


Fig. 570.

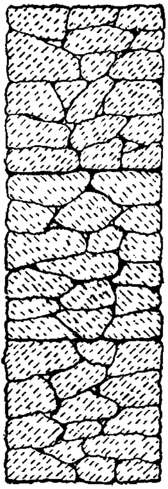


Fig. 571.

Figs. 570 and 571.—Random Rubble brought up to Courses with Block and Start Quoin.

reach within 1 in. of the edges of the stones, is thought to equalise the pressure over the beds by yielding to any slight irregularities on them. However, experiments made by Mr. Kirkaldy have shown that the use of lead instead of mortar is a great mistake. He found that stones bedded on thin pieces of pine, instead of lead, equal in area to the bed-joint, bore a greater crushing force than stones of double their sectional area bedded on lead in the usual way. The lead which had been used showed no signs of accommodating itself to the irregularities of the beds" (Seddon's "Builders' Work"). It seems remarkable that the lead should not adapt itself to the irregularities of the stone in this case, as the pressure might be expected to cause the lead to flow and burst off an outer ring of stone as it were by hydraulic pressure. The piece of pine would be liable to decay if used in construction. The special precautions to be taken in bedding the stones forming the shaft of a column are : (1) To set the stones on their natural bed. (2) To work the beds true and out of winding. (3) To insert a central dowel accurately fitted and of sufficient strength. (4) If a jointing material is used, it

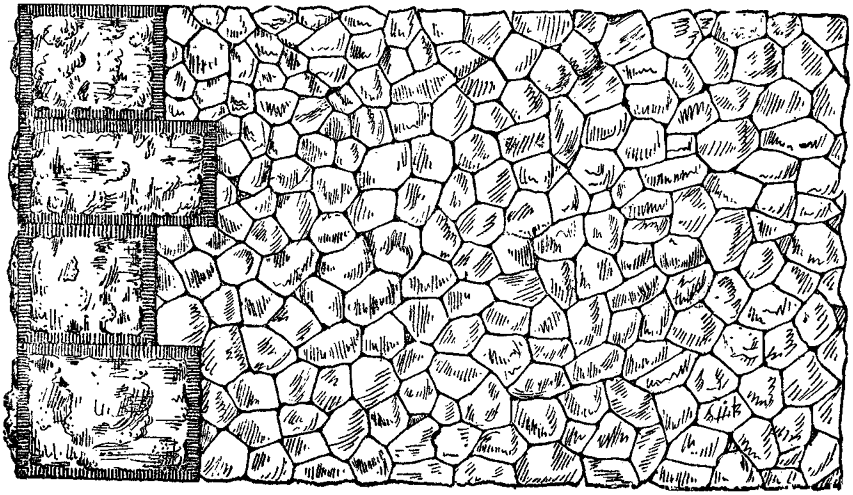


Fig. 572.—Rustic Rubble or Polygonal Ragwork and Rock-faced Quoin with Drafted Margins.

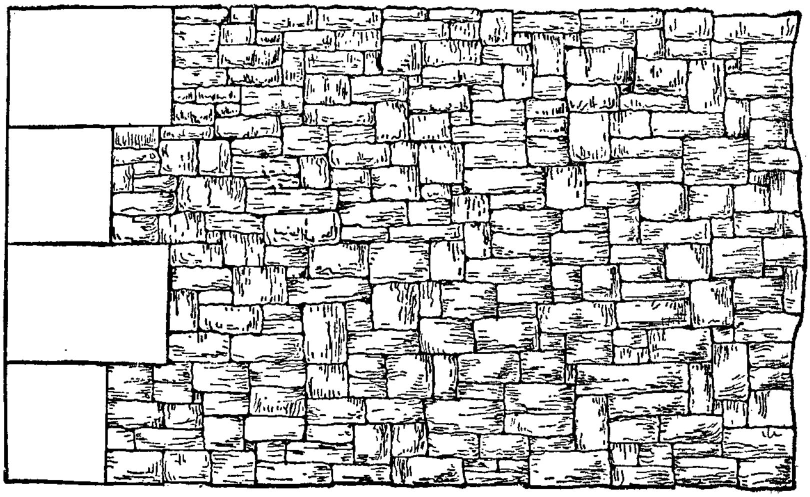


Fig. 573.—Squared Random Rubble, or Random Coursed Rubble, with Ashlar Quoin.

should be neat Portland cement, or cement and sand, the sand being screened by hand through a fine sieve.

Stone Walling.

The principal kinds of stone walling include: Flat bedded rubble laid dry, with a rubble-on-edge fencing (Fig. 568); uncoursed or random rubble with brick quoin and pier (Fig. 569); random rubble brought up to courses with block and start quoin (Figs. 570 and 571); rustic rubble or polygonal ragwork and rock-faced quoin with drafted margins (Fig. 572); squared random rubble, or random coursed rubble, with ashlar quoin (Fig. 573); squared random rubble brought up to courses (Fig. 574); squared or coursed rubble with Gothic coping and lead plug (Figs. 575 and 576); block in course or large squared rubble (Fig. 577); and ashlar work with saddle-back coping and iron cramp (Figs. 578 and 579). All these illustrations are to a scale of $\frac{3}{8}$ to $\frac{1}{2}$ in. = 1 ft.

Building up Rubble Walling.

Fig. 580 shows the elevation of two courses of rubble masonry (the course being, say, 15 in. deep). The stones in the second course are laid on the broadest face with the natural bed horizontal and placed so as to cover the joints in the course below; the spaces between are

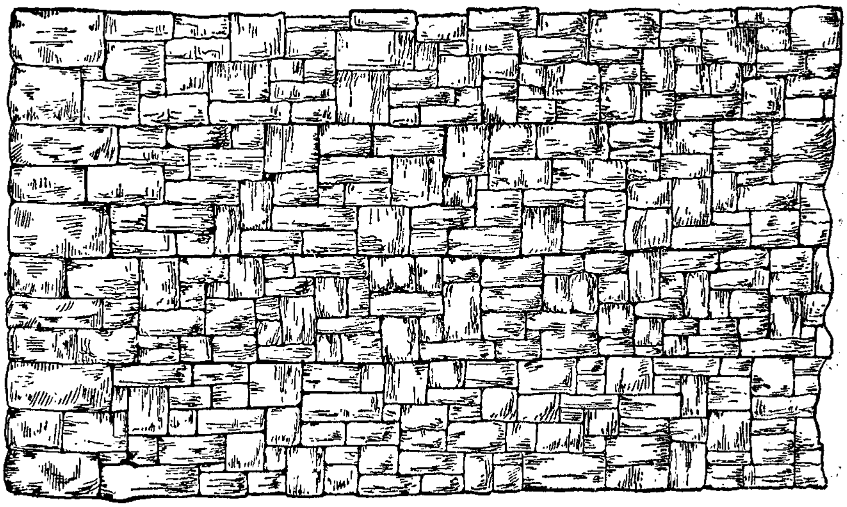


Fig. 574.—Squared Random Rubble brought up to Courses.

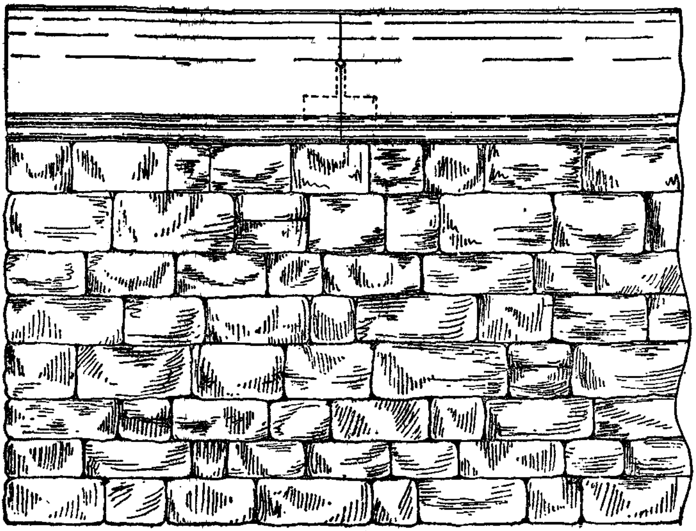


Fig. 575.

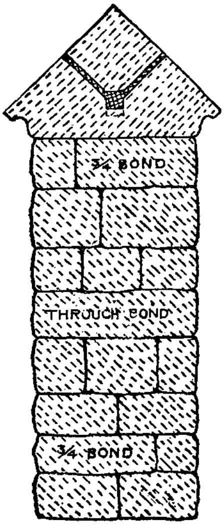


Fig. 576.

Figs. 575 and 576.—Squared or Coursed Rubble with Gothic Coping.

then filled up with other pieces of suitable size, so that the course may finish level. The numbers in the second course indicate the order in which the stones are laid. The masonry might be described as "hammer-dressed rubble masonry brought up to 15-in. courses." The work is kept in line by means of line pins and a cord stretched over them, the quoins being built in advance of the other work so that they may hold the line pins and form a gauge for the remainder of the work. In rock-faced work (see the section, Fig. 581) the plumb rule is held as near as the rough face of the blocks will permit, and the distance measured with a 2-ft. rule. Or a long French nail may be driven near the bottom to bear against the edge of a lower course, and the plumb of an upper course may be measured by a 2-ft. rule.

Quoins and Angles of Openings in Rubble Walling.

Of whatever kind the walling may be, the quoins and the angles of the openings should be of better quality stone, be more carefully squared and bedded, and be arranged to bond alternately long and short in both directions. Walls 18 in. thick should have to every square yard at least one good bonder back and front, going at least 12 in. into the wall. This is better than having through or thorough bonds, which are apt to transmit moisture to the inside of the wall.

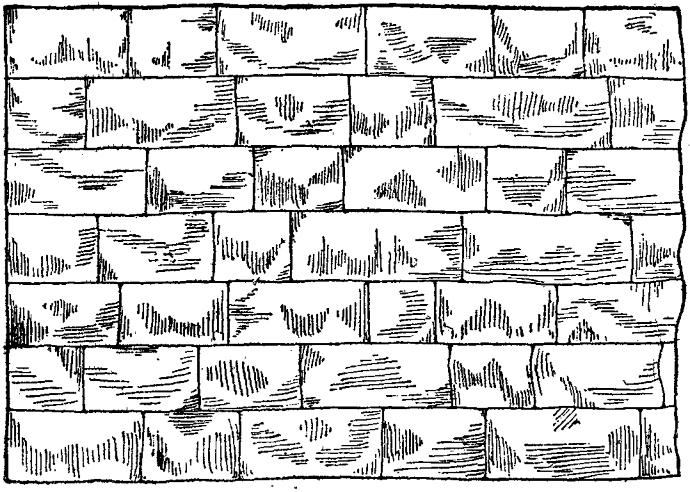


Fig. 577.—Block in Course or Large Squared Rubble.

Bonding Stone Walling: Jumpers.

Referring to the class of work in which large stones or "jumpers" are used, Colonel Seddon says:—"In irregular coursed rubble the stones are bedded flat and run to a certain extent in courses, which are, however, stopped at irregular intervals just as any deeper stones happen to break them. The joints in this case are not necessarily vertical, but in a superior class of the work all the joints are specified to be vertical and the beds horizontal, by which the more regular appearance of squared rubble is

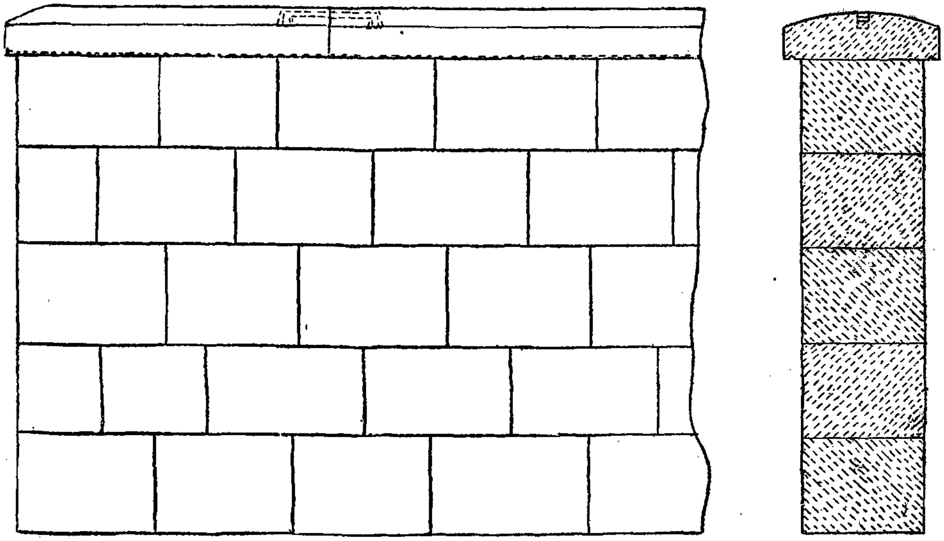


Fig. 578. Fig. 579.

Figs. 578 and 579.—Ashlar Work with Saddle-back Coping.

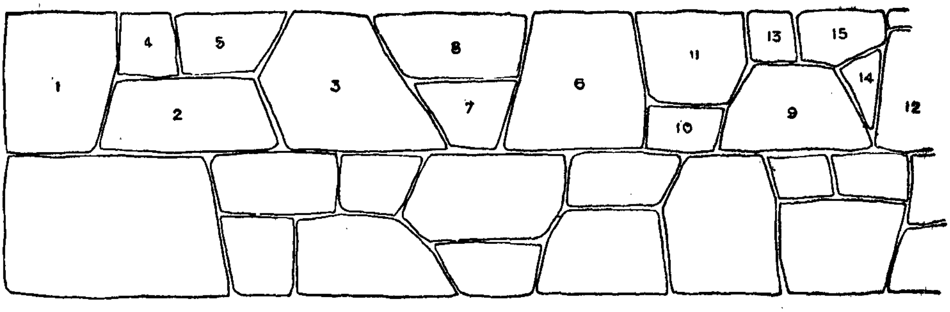


Fig. 580.—Two Courses of Rubble Wall, showing Order of Laying Stones.

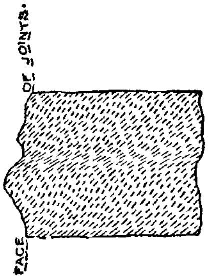


Fig. 581.—Section of Rock-faced Stone.

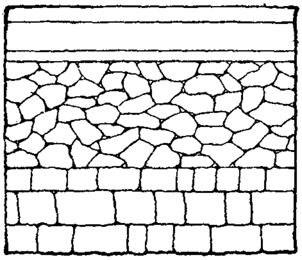


Fig. 582.—Wall with Coursed Rubble, Random Rubble, and Coping.

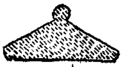


Fig. 583.—Section of Saddle-back Coping with Roll.

obtained. The stones are all shaped, roughly squared if required, and hammer or axe-faced, in the gross, ready for the waller to set in the wall. This class of masonry is very bold and effective, especially for engineering purposes, when roughly squared and hammer-dressed on face, and built up with a good proportion of large stones." Professor Rankine says, in addition :—"One fourth part at least of the face in each course should consist of bond stones or headers ; each header to be of the entire width of the course, of a depth ranging from one-and-a-half times to double that depth, and of a length extending into the building to from three to five times that depth, as in ashlar. Those headers should be roughly squared with the hammer, and their beds hammer-dressed to approximate planes; and care should be taken not to place the headers of successive courses above each other, for that arrangement would cause a deficiency of bond in the intermediate parts of the course. Between the headers each course is to be built of smaller stones, of which there may be one, two, or more in the depth of the course. These are sometimes roughly squared, so as to have vertical side-joints ; sometimes the stones are taken as they come, so that the side-joints are irregular ; but no side-joint should form an angle with a bed-joint sharper than 60°. Care should be taken not only that each stone shall rest on its natural bed, but that the sides parallel to that natural bed shall be the largest, so that the stone may be flat, and not be set on edge or on end. Howsoever small and irregular the stones may be, care should be taken to make the courses break joint. Hollows between the larger stones should be carefully filled with smaller stones completely embedded in mortar."

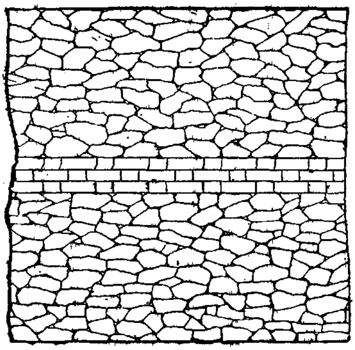


Fig. 584.—Random Rubble Wall with Brick Lacing Course.

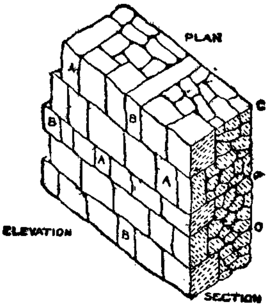


Fig. 585.—Rubble Wall Faced with Coursed Rubble.

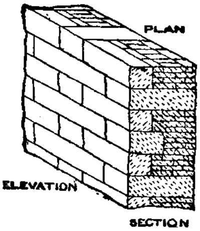


Fig. 586.—Brick Wall Faced with Ashlar.

Composite Walling.

Under this heading it is convenient to give some notes on walls containing individually more than one style of arrangement or more than one principal material. For instance, Fig. 582 gives an elevation of part of a 16-in. stone enclosure wall, showing at the bottom coursed rubble, higher up random rubble, and at the top saddle-back stone coping, with a 3-in. roll (see also Fig. 583). The depth of a stone should not exceed its bed in best work and

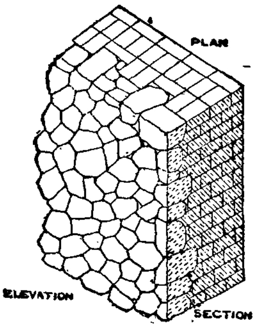


Fig. 587.—Brick Wall Faced with Rubble.

the construction shown in Fig. 582 can therefore be improved upon slightly. Fig. 584 is the part elevation of a stone wall built in random rubble, with a lacing course of three courses of bricks laid in Flemish bond. In the case of Fig. 585, which shows a rubble wall faced with coursed rubble, the rubble backing must be bedded as firmly as possible, with thin joints, and must be "brought up to courses" at frequent intervals. Three-quarter bonds and occasional through bonds must be inserted in the facing to tie in the backing. All stones must be laid with their broadest face downwards. In Fig. 585, A shows three-quarter bond stones, B through bond stones, and C the level courses. In a brick wall faced with ashlar (Fig. 586) the depth of the courses of ashlar must be the same as a given number of courses of the brickwork, so as to admit of proper bond stones. The joints of the brickwork should be as thin as possible to reduce the settlement, and it is better to build in cement to avoid settlement altogether.

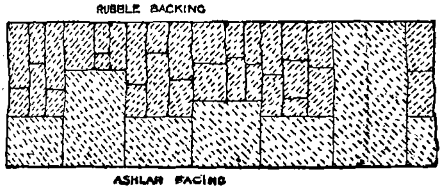


Fig. 588.—Rubble Wall with Ashlar Facing.

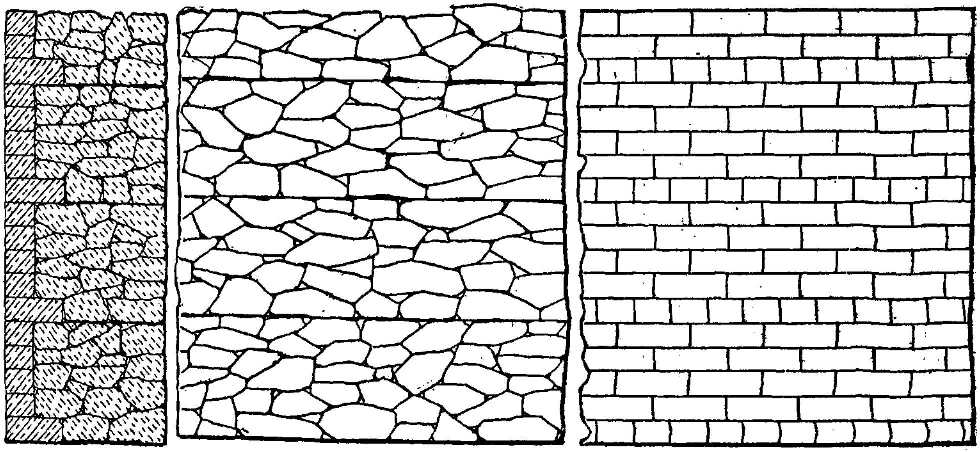


Fig. 589. Fig. 590. Fig. 591.

Figs. 589 to 591.—Section, Front Elevation, and Back Elevation of Coursed Rubble Wall Lined with Brickwork.

General instructions for the above are: Lay the stone as far as possible on its natural bed. See that beds of each stone are level for at least three-fourths of area. Keep joints of backing as thin as possible, and with rubble backing fill in with spalls to all open joints.

Brick Wall Faced with Ashlar.

The following is the specification of work to be executed in building brick wall faced with ashlar. General conditions :—Materials: Brickwork to be of sound, well-burnt stock bricks, uniform in size and arranged in English bond, of which no four courses to rise more than 12 in., set in stone lime mortar mixed in the proportion of one to three with clean sharp sand. Ashlar to be of Portland stone (Whitbed) set in very fine stone lime mortar free from grit, and the outer edge to be filled with lime putty to a depth of $\frac{3}{8}$ in., and all stones to be carefully dressed. Fig. 588 is a section of rubble wall with ashlar facing, the rubble being brought up to level courses with the ashlar work to secure efficient bonding. Figs. 589 to 591 show section and elevations of part of an external wall of random rubble worked up to courses and finished inside with a half-brick lining. When a brick wall is faced with Kentish rag (see Fig. 587) the stone, being very tough, is usually dressed with the hammer into rough polygonal blocks. Bond stones may be inserted at intervals; but owing to the difficulty of meeting the courses, bonding by galvanised iron cramps is more reliable.

Specification for Flat-bedded Rubble Walling Built up to Courses and Lined with 4½-in. Brickwork.

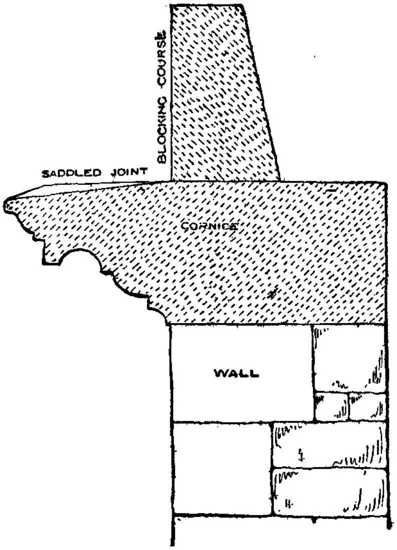


Fig. 592.—Section of Cornice.

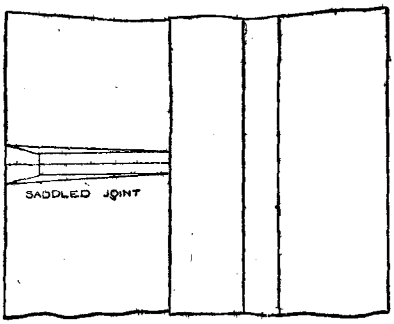


Fig. 593.—Plan of Cornice, showing Saddled Joint.

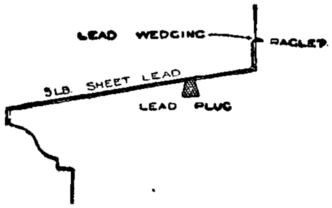
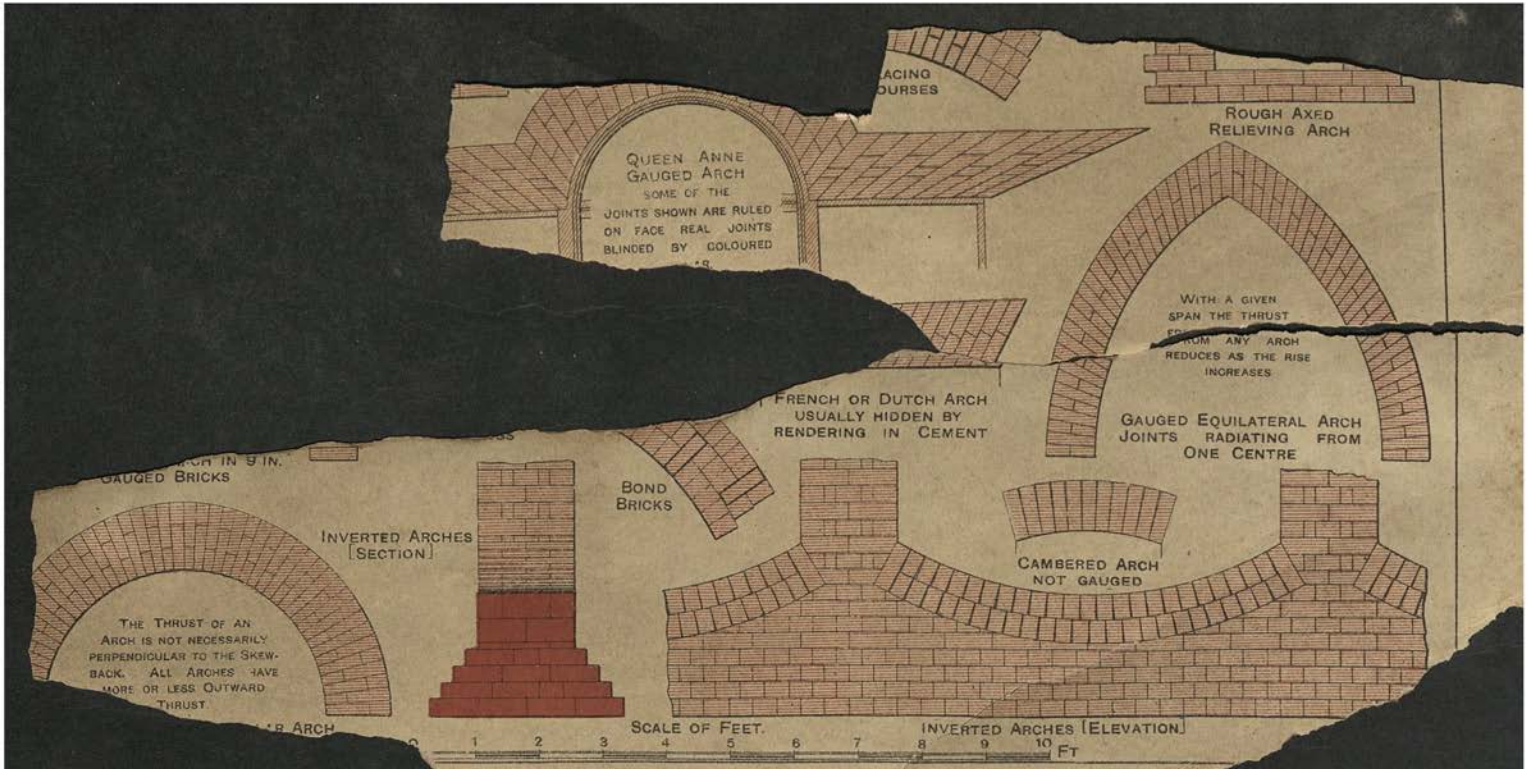


Fig. 594.—Lead Covering of Cornice.

General conditions as usual, and particularly . . . . All scaffolding, tools, tackle and other plant, water, labour, and materials to be provided for the efficient carrying out and completion of this contract. The whole to be done in a workmanlike manner to the complete satisfaction of the architect. *Excavator.*—The excavating to include strutting and planking if necessary, part of the earth to be returned and rammed, and the remainder carted away. The concrete to be composed of 1 part fresh ground stone lime to 5 parts of clean ballast and 1 part of sharp sand. The trenches to be excavated to the depth and width shown on drawings, and filled to a depth of 1 ft. with concrete, as above described, well rammed and brought to a level surface to receive footings of wall and project 6 in. beyond them. The earth to be filled in and well rammed round footings as the work proceeds. *Bricklayer.*—The bricks to be sound, hard, square, well-burnt stocks, free from all defects, and approved by the architect. The mortar to be composed of 1 part grey-stone lime to 3 parts clean sharp sand, well mixed, sufficient for one day's use only to be mixed at one time; or 1 part Roman cement to 1 part sand, mixed as used, up to and including damp-proof course. *Mason.*—The rubble walling to be of the best . . . or other approved flat-bedded rubble stone, roughly squared, with hammer-dressed outer face, 14 in. thick, and brought up to level courses with a backing of 4½-in. brickwork in stretching bond, the same to be well and properly bonded into the stonework by headers every 18 in. in every fourth or fifth course where level with the rubble stone. The stonework to be set close up to brickwork, the joints well flushed up with mortar, and grouted every level course, and pointed externally with a neat cement joint. No filling with spalls nor wedging up will be allowed, and a sufficient number of bond stones must be inserted, reaching from the face to the brick backing. The stone for the dressings is to be . . . the best of its kind, well seasoned, free from all vents, shakes, sand-cracks, or other defects. To be properly worked, set on its natural bed with all necessary slate dowels, double plugs, cramps, lead, etc. To be well and properly bonded to the brickwork and rubble walling, and jambs to have not less


ACING
OURSES
ROUGH AXED RELIEVING ARCH
QUEEN ANNE GAUGED ARCH
SOME OF THE JOINTS SHOWN ARE RULED ON FACE REAL JOINTS BLINDED BY COLOURED
WITH A GIVEN SPAN THE THRUST FROM ANY ARCH REDUCES AS THE RISE INCREASES
FRENCH OR DUTCH ARCH USUALLY HIDDEN BY RENDERING IN CEMENT
GAUGED EQUILATERAL ARCH JOINTS RADIATING FROM ONE CENTRE
IN 9 IN.
GAUGED BRICKS
BOND BRICKS
INVERTED ARCHES [SECTION]
CAMBERED ARCH NOT GAUGED
THE THRUST OF AN ARCH IS NOT NECESSARILY PERPENDICULAR TO THE SKEW-BACK. ALL ARCHES HAVE MORE OR LESS OUTWARD THRUST.
ARCH
SCALE OF FEET.
INVERTED ARCHES [ELEVATION]
0
1
2
3
4
5
6
7
8
9
10 FT

BRICK ARCHES.

than 1 bond stone to every 3 ft. in height. The whole to be worked and rubbed according to drawings and set in fine mortar and pointed in cement. The bed joints of jambs to have 1 in. by 1 in. by 3 in. slate dowels, and all joints in string courses, plinths and copings to be pebble plugged and run with cement where not otherwise cramped together. A damp-proof course of three layers of slates breaking joint is to be bedded in cement and laid throughout the full thickness of the wall above ground line and below ground floor wall plate (or Brunswick rock asphalt, ¼ in. thick, may be used instead). Air bricks and drains to be inserted immediately above the damp-proof course.

Definition of "Shiner."

The question "What is a shiner?" set in the South Kensington Building Construction examinations, 1901, raised a considerable discussion at the time. The writer believes that a "shiner" is a local or slang term used to express the appearance of a stone laid with its natural bed exposed as a face, generally so laid because this is the largest side, and the object is to make the stone reach higher, as in a jumper in snecked rubble work, or to look substantial when the heart of the wall is made up with smaller stone; but the term can hardly apply to granite and trap, which have no natural bed. Other definitions that have been advanced are:—(*a*) A marble or granite polisher; (*b*) a stone set with its face bedded and generally with less depth of bed than height of face; (*c*) a piece of limestone with a smooth, shiny face, found in rubble masonry walling usually covered with cement roughcast work; when the cement falls off, as it is very apt to do, the shiners have to be cut back for 2 in. or 3 in. and filled with cement work to form a key for the new cement face work; (*d*) in coursed rubble, a stone breaking through the height of two or more courses, and having insufficient bed: (*e*) any stone having a bottom bed of less width than one-third the height of the stone.

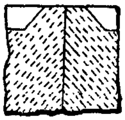


Fig. 596.—Section of Saddled Joint.

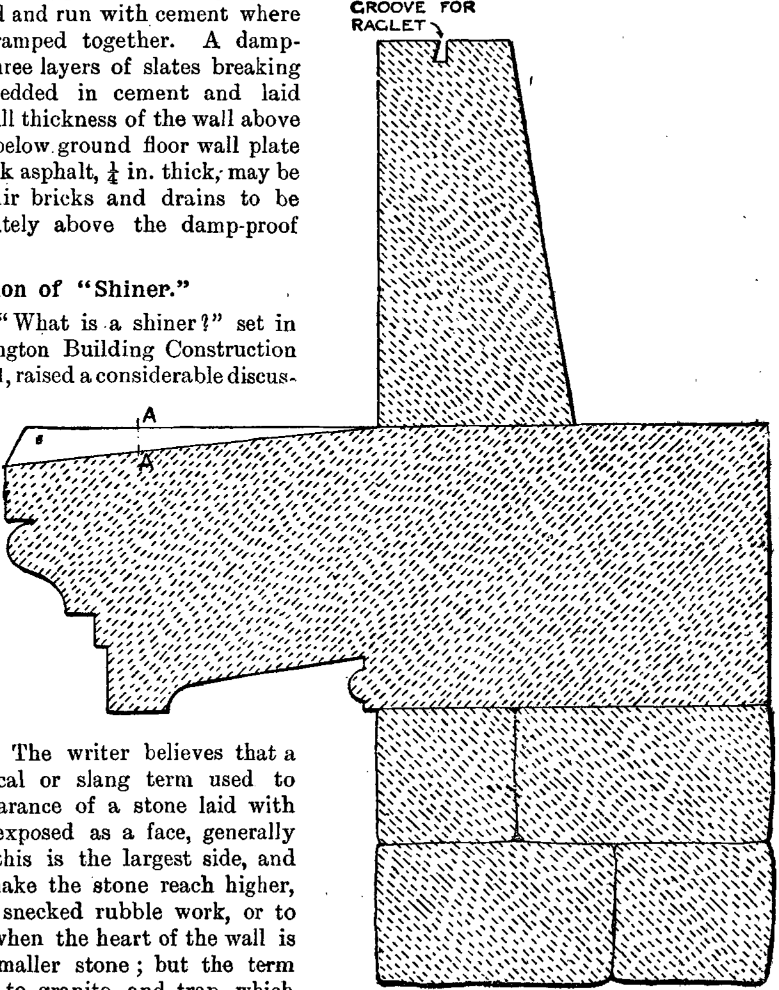


Fig. 595.—Section of Stone Cornice on Coursed Rubble Wall.

Setting Cornice on Stone Wall.

The precautions to be adopted in setting the stones of a cornice are—(1) A sufficient bulk of the stone should rest on the wall to balance the overhanging part, and this should be further assisted by the weight of the blocking course. (2) The layers or stratification of the stone should be vertical and perpendicular to the face of the work. If laid on its natural bed parts of the moulding would be apt to drop off in frosty weather. (3) If the stone has a tendency to show open laminations the weathered part should be covered with 5 lb. sheet lead turned up against blocking course in a raglet or groove, and soldered down to plugs at intervals. Fig. 592 shows the section of a cornice projecting 14 in. from the face of the wall; the stratification of the stone is vertical, and runs from front to back in this course, and horizontal in the blocking course. Fig. 593 shows a saddled joint between two of the cornice stones to carry the rain away from the joint; the elevation of this joint is shown in Fig. 592. Fig. 594 shows how the lead is fixed for protecting the stone when it is of a flaky nature and liable to weather unequally. For cornices, saddle joints and blocking courses, see also Fig. 552 (p. 140) and Figs. 595 and 596.

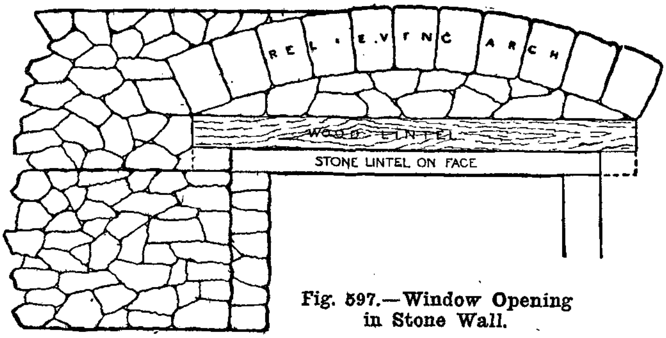


Fig. 597.—Window Opening in Stone Wall.

Window Opening in Stone Building.

The internal elevation of th[illegible] a window opening in [illegible] one building is shown [illegible] 597. There is a wood lintel at back, a stone lintel on the face and above is a relieving arch. The rough rubble wall is built up to 20-in. courses.

Centering for Masonry Arch.

The centering for a 27-ft. span semicircular masonry arch is shown by Figs. 598 and 599; description is unnecessary after the detailed explanations of centerings for brick arches given in the previous section.

Centering for Gothic Vaulting.

The centering for Gothic groined vaulting as shown in the plan (Fig. 600) should be secured at all junctions, and, when in place, covered by laggings. A (Fig. 601) shows a main centre diagonally across the vault; B (Fig. 602) shows the two half centres to make up the other diagonal; C (Fig. 603) shows four intermediate centres to support the laggings between the main centres; D (Fig. 604) shows the centres for supporting the four arches on the sides of the vault as indicated in the plan (Fig. 600). The letter references in Figs. 601 to 604 agree with those in Fig. 600. Angle plates should be

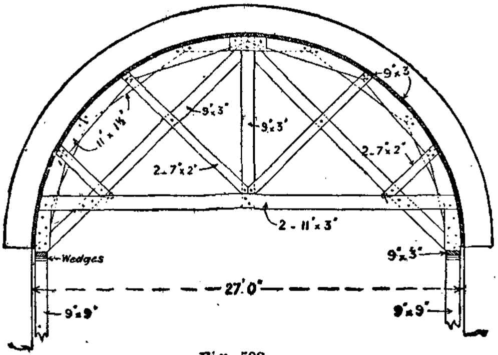


Fig. 598.

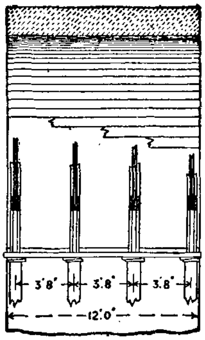


Fig. 599.

Figs. 598 and 599.—Centering for Semicircular Arch.

used for the purpose of connecting the different centres, and support should be placed under each end and at each separate junction. The centres under the ribs of the vaulting must be of such dimensions as to support the ribs, and pieces nailed on the sides of these centres will carry the laggings for the vaulting as shown in Fig. 605.

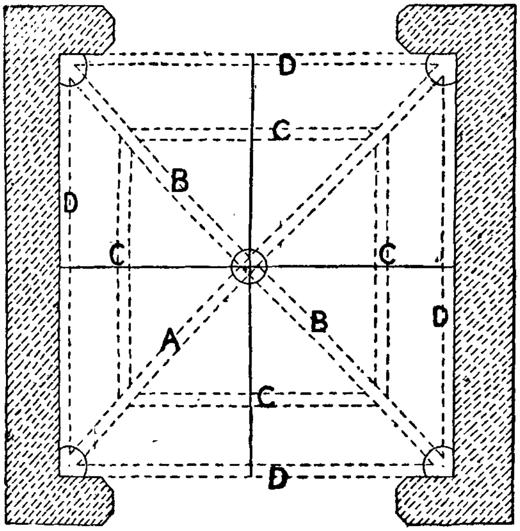


Fig. 600.—Plan of Centering for Gothic Vaulting.

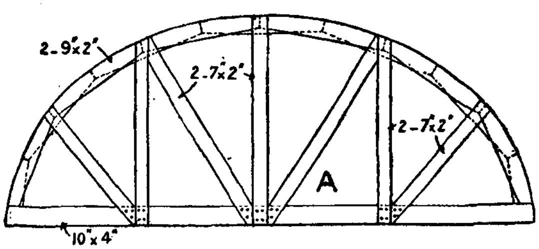


Fig. 601.—Main Centre Diagonally across Vault.

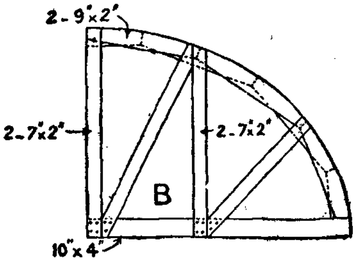


Fig. 602.—Half-centre for Vaulting.

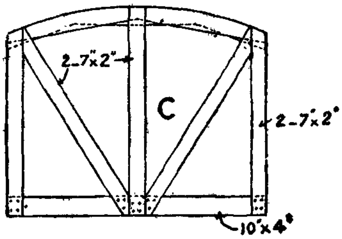


Fig. 603.—Intermediate Centre for Vaulting.

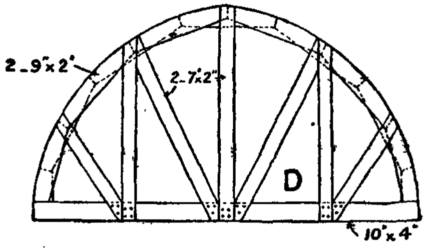


Fig. 604.—Centre for Arches at Sides of Vault.

Setting Out and Measuring Groined Vaulting.

To find the elevation of a groin in unequal vaulting, draw a plan where the intersections take place; *a b* and *c d* (Fig. 606) represent the 30-ft. span vaulting, and *b c* and *a d* represent the 25-ft. span vaulting, both with the same rise of 5 ft. Upon *a b* and *b c* draw sections of the respective vaults, and parallel with each base draw equidistant lines to meet the curves. From each of the points on the curves draw perpendicular lines which by their intersection will give the groin *b d*. In the given example this will not be distinguishable from an ordinary mitre line, but under certain conditions it may be a curved line. Now parallel with *b d* draw *e f* as the base of elevation of groin, and project lines at right angles from the intersecting points on the groin line, cutting them off by lines parallel to *e f* at the same distance apart as before. Then *e g f* will be the elevation of groin. For the measurement of the work, probably the simplest way would be as follows, but it would depend very much upon the district, the stone, and the object of the vaulting:—Cub. ft. "——stone" in barrel vaulting including all labours in sunk beds and joints, measured net. Note—The rough cube stone will average 50 per cent. over this quantity. Ft. sup. circular work sunk in soffit of vaulting. Ft. run extra labour and waste at groins of vaulting.

Doorway of Public Building.

A doorway in Perpendicular masonry is illustrated by Fig. 607; sections taken on the lines A A and B B respectively are given by Figs. 608 and 609.

Stone Staircase for Hall of Public Building.

A sketch elevation of part of a stone staircase suitable to the hall of an important public building is presented by Fig. 610. Figs. 611 to 614 show explanatory details, including a vertical cross section of two steps and a landing. Specification.—*Mason.* The whole of the stone to be of the best description of its respective kind, to the architect's approval, to be free from sand holes, vents, and all other defects; to be worked to lie on its natural bed when set, and to be bedded and jointed in putty with fine joints which are intended to show. All the stone to be worked on the site, and particular care is to be taken to preserve all the joints of the stonework from the irregular appearance which is caused by the arrises being broken before the stones are set. No work thus injured will be allowed to be used, and no patching will be allowed. The stonework to be so truly worked as not to require any cleaning off beyond washing. The staircase to be of basebed Portland stone, rubbed all round, spandril steps in three lengths splay-rebated and splay back-jointed with sunk and moulded fronts and solid square ends built into walls, 11 in. from face to face of riser and 5½-in. rise. The bottom step to be solid, with curtail ends as shown. The landing to be 4 in. thick, and joggle jointed as shown. Provide models to the approbation of the architect, made by an artist in London, for the whole of the carving and sculpture; the whole to be made to a scale of 3 in. to 1 ft. *Smith and Founder.*—The stone staircase to be supported by No. 2 8-in. by 4-in. by 19-lb. rolled steel joists from an approved maker, bent to shape and fitted with 12-in. by 6-in. double cover plates bolted as shown. The upper ends to be built into wall at back of landing. The lower ends to pass through ground floor paving and to be attached to main girder. The joists to be supported at their junctions by 6-in. cast-iron columns, and secured by two bolts. The columns to be of ¾-in. metal of the best quality soft grey pig-iron from the second melting, and to be free from cracks, flaws, cold shuts, blisters, sand holes, or any other defects; and the metal to be of an even thickness throughout.

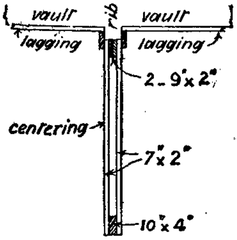


Fig. 605.—Laggings for Vaulting.

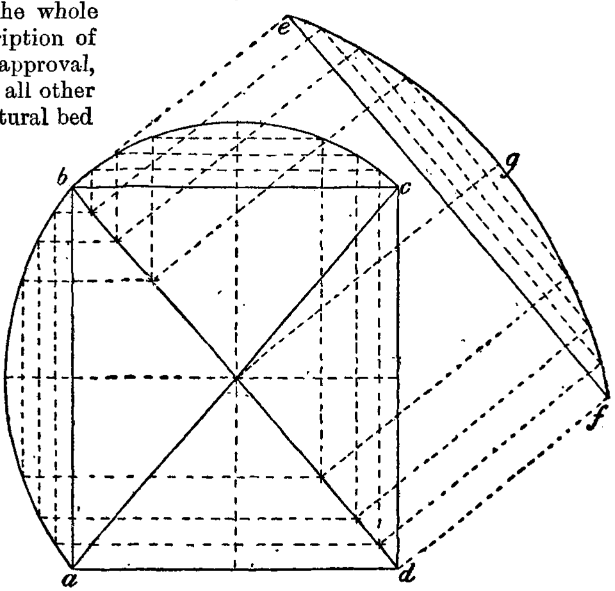


Fig. 606.—Setting Out Groined Vaulting.

Preserving Surfaces of Masonry.

Masonry stone surfaces may be preserved from the action of the air by the following means:—(1) Those that are of the nature of a paint, stopping or varnish; (2) those that are mechanical in their operation; (3) those that operate partly mechanically and partly chemically. The objections to these methods are as follows: First.—(*a*) They change the natural colour and hide the natural grain of the stone, giving to it an artificial appearance; (*b*) they imprison in the stone the moisture, which is sure to be drawn into it through its untreated surfaces, and thus increase the risk of fracture by frost; (*c*) they do not adhere well to the stone, and, becoming oxidised by exposure to the air, have to be periodically renewed. Second.—These usually consist of the employment of silica in the form of a wash, the silica filling the pores and becoming solidified there. But this method is attended with serious disadvantages, for while it fills the pores of the stone with a foreign substance which is itself durable, it leaves the walls of the pores unaffected, and as the wasting away of these walls under atmospheric influences enlarges the pores, the foreign bodies are loosened and fall out, and the protection to the stone is gone. Third.—Few of these have secured the attention of practical men; they may be described as applications of alkaline silicates, which introduce into the stone salts that are soluble—herein lies the drawback. The salts, being soluble, impregnate the mass of the stone, and throw out upon its surface a white film resembling hoar frost, which permanently disfigures the building. And they do worse than this; it is their nature to attract and draw into the stone moisture from the air. By this action there is set up in the stone the process known as nitrification—that is, the stone has imparted to it the qualities of nitre. Potash salts are deposited and become the fertiliser of vegetable growth, and the stone, instead of being improved, is injured. Other methods there are which are too expensive to be really useful. So that apparently none of these methods is quite successful. The prime conditions which are essential to practical success in any endeavour to increase, permanently, the resistance of building stones to atmospheric action are four in number—namely (1) the process should be largely chemical; (2) whatever change is effected in the stone, the resultant products should be themselves insoluble; (3) the process should not alter the natural appearance of the stone; (4) its cost should be moderate. The effectual remedy seems to be in the use of "Fluate" or the process known as fluosilicatisation, discovered by an eminent French chemist—M. Kessler,

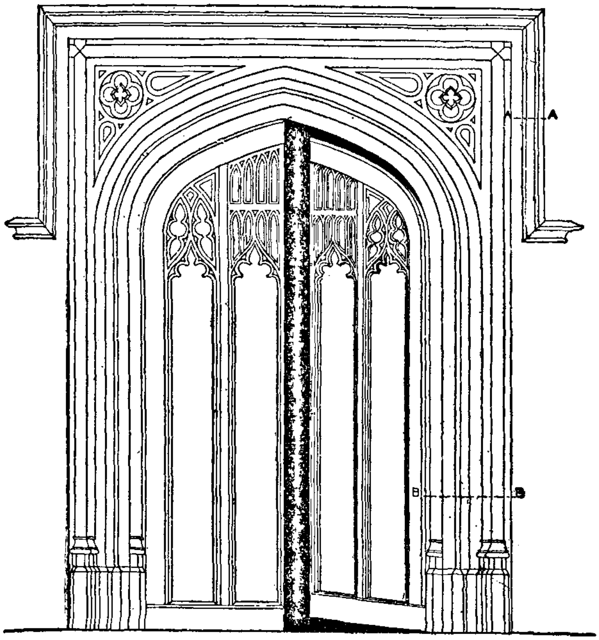


Fig. 607.—Doorway of Public Building.

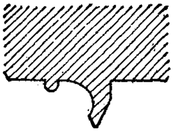


Fig. 608.—Section of Hood Moulding round Doorway.

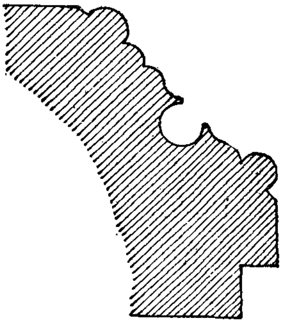


Fig. 609.—Section of Jamb Mouldings.

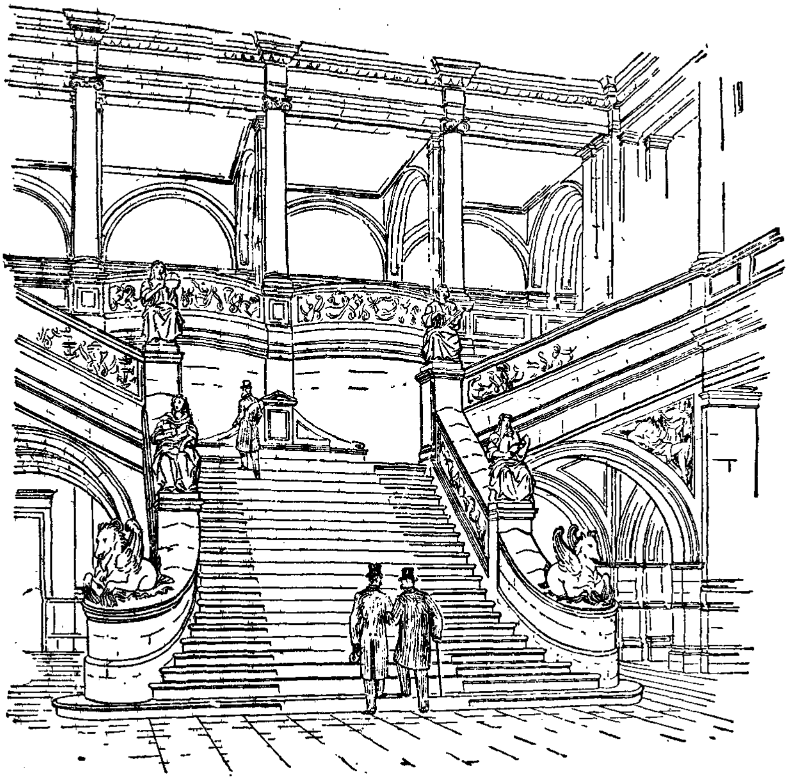


Fig. 610.—Stone Staircase for Hall of Public Building.

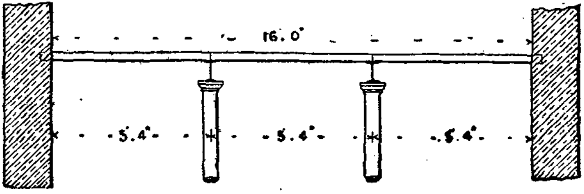


Fig. 611.—Part Vertical Section of Staircase, showing. Iron Columns.

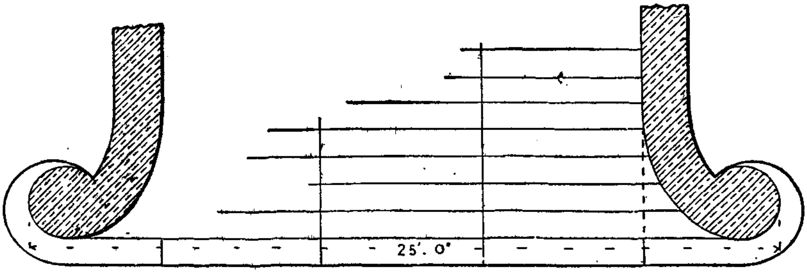
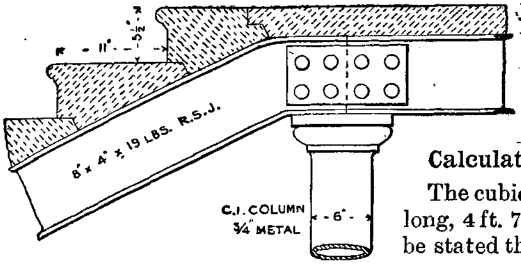


Fig. 612.—Part Horizontal Section of Staircase.

of Clermont-Ferrand. The agents (The Bath Stone Firms, Ltd., Abbey Yard, Bath) for this fluating process, or, rather, for the necessary material, affirm that their process conforms with all these conditions and has many additional advantages, such as resistance to acids, prevention of vegetation, no skilled labour required, no expensive scaffolding, may be made to penetrate any depth, no unequal expansion and contraction, pores of stone not hermetically sealed, rain kept out, capillarity test very satisfactory, injury by frost prevented, stone kept comparatively clean, permanence of effects produced, will not in course of time prove useless or injurious, hardens floors, steps, and prevents stone dust rubbing off, etc. etc. The process has been used very largely in France, Switzerland, Italy, Austria, and other Continental countries, with great success.



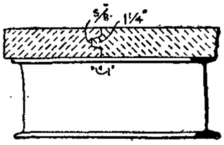


Fig. 613.—Vertical Section of Landing and Steps.

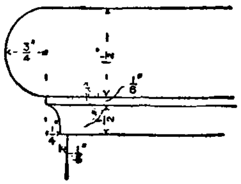


Fig. 614.—Detail of Moulding on Stone Step.

Measuring Irregular Stone Flagging.

The general rule in measuring irregular stonework is to measure the smallest rectangular figure out of which the flag can be cut (as in Fig. 615), except sometimes in the case of feather-edged steps, where two are cut out of one block and are so described. When there is a large number of pieces of the same size and shape, the items should be given as follows: 1,054 ft. sup. 2-in. York paving measured square in No. 248 flags, as sketch (Fig. 616); 1,364 ft. run coped edges to 2-in. York paving; 496 ft. run circular work on edge of ditto; 248 ft. run circular sunk work on edge of ditto.

Calculating Contents of Slab of Stone.

The cubic contents of a slab of stone 5 ft. 3 in. long, 4 ft. 7 in. wide, and 2 ft. 9 in. thick, would be stated thus :

ft. in.	*ft. in.*	
5 3 4 7 2 9	66 2	cube stone,

and would be worked out by a combination of duodecimals and practice, thus :—

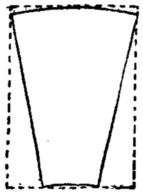


Fig. 615.

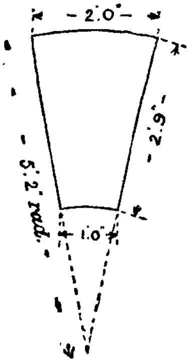


Fig. 616.

Figs. 615 and 616.—Diagrams of Stone Flags.

	ft.	*in.*	
	5	3	
	4	7	
5 ft. 3 in. × 4 =	21	0	
6 in. = ½ of 5 ft. 3 in. =	2	7	6
1 in. = ⅙ of 6 in. =		5	3
	24	0	9
	2	9	
	48	1	6
6 in. = ½ of 24 ft. 0 in. 9 =	12	0	4½
in. = ½ of 6 in. =	6	0	2¼
	66	2	0¾

LIMES, MORTARS, AND CEMENTS.

Limes Used in Buildings.

Limes for building purposes are obtained by the calcination of chalk, limestone, and other calcareous minerals and rocks, the effect of which is to drive off the carbonic acid and moisture, leaving quick or caustic lime. Limes can be distinguished by their condition, colour, feel, and slaking and setting properties.

Burning Lime.

The formula for calcic carbonate is $CaCO_3$; the atomic weights being Ca 40, O 16, C 12, the weight of unit mass of pure carbonate of lime will be $40 + 12 + 3 \times 16 = 100$. Limestone in burning produces carbon dioxide CO_2, caustic lime CaO, watery vapour H_2O, and inert matter present as impurities.

Thus 3·424 tons CaCO =

$$\begin{array}{lll} \frac{40}{100}\text{ Ca} = 1\cdot3696 & \text{tons} & \text{Ca.} \\ \frac{12}{100}\text{ C} = \cdot41088 & ,, & \text{C.} \\ \frac{3 \times 16}{100}\text{ O} = 1\cdot64352 & ,, & \text{O.} \\ \underline{\underline{3\cdot42400}} & & \end{array}$$

Resulting products—

$$\begin{array}{rll} 1\cdot3696 & \text{tons} & \text{Ca.} \\ \underline{\cdot54784} & ,, & \text{O} \\ 1\cdot91744 & ,, & \text{CaO.} \\ \cdot41088 & ,, & \text{C.} \\ \underline{1\cdot09568} & ,, & \text{O.} \\ 1\cdot50656 & ,, & CO_2 \end{array}$$

Therefore the resulting weight of pure lime from a charge of 4 tons of limestone would be 1·91744 tons, or adding the 0·4 of inert matter =2·31744 tons of commercial lime.

Lime-Kilns.

Tunnel Lime-Kiln.—A tunnel kiln (Fig. 617), called also a "continuous running," "perpetual," or "draw-kiln," is shaped internally like a cylinder, an inverted cone, or pair of vertical cones base to base. It is lined with firebrick with hollow space behind, and has an opening below, generally protected from the weather by a shed to preserve the fresh lime as it is produced. At the lower extremity of the cone is a grating of loose fire-bars, on which is placed a layer of brushwood, and then alternate layers of coal and moistened stone reaching to the top, the largest pieces being in the middle where they will get most heat. As the lime becomes burnt it is withdrawn through the grating, and fresh stone and fuel are added at the top, the layers of stone being about six times the thickness of the layers of fuel. The form shown is frequently erected as a temporary arrangement to burn the lime during the progress of large works. The kiln may be built of bricks, stone, or concrete, or it may be sunk in the ground as shown by Fig. 618. A lime-kiln is usually built high enough to ensure

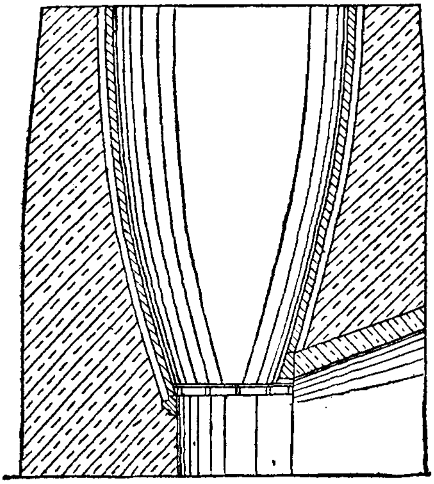


Fig. 617.—Tunnel Lime-Kiln.

that the limestone shall be sufficiently long in contact with the hot fuel and attain a temperature that will drive off the carbon dioxide; anything more than this would be waste. The size shown—namely, 16 ft. high, 4 ft. wide at bottom, and 9 ft. wide at top—will hold about 25 tons of limestone, and will burn sufficient lime to keep twenty bricklayers constantly supplied with mortar. The preparation of the limestone for burning is to break it up into pieces not larger than one man can carry, to sprinkle them with water before putting them into the kiln, to put the smaller pieces in the bottom, and put in the larger pieces when the kiln is well started.

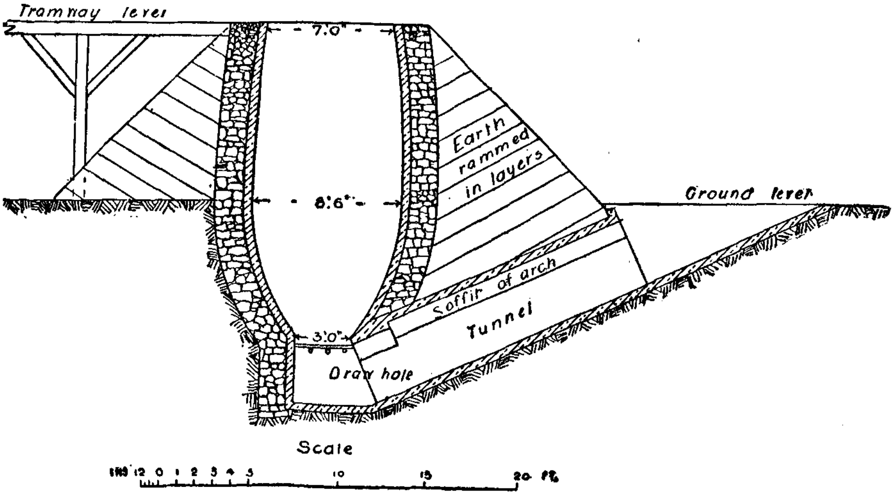


Fig. 618.—Sunk Tunnel Lime-Kiln.

Flare Lime-Kiln.—In working a flare kiln, Fig. 619, the limestone is packed in over a rough arch formed of large pieces of the stone. The fire is lighted in the space left below, the flame alone being in contact with the stone. In a tunnel kiln, fuel and stones are placed in alternate layers, the largest pieces being placed in the middle, where they will get most heat. As the lime becomes burnt it is withdrawn through the grating, and fresh stone and fuel are added at the top. The advantages of the flare kiln over the tunnel kiln are ease of management and uniformity of lime produced. The disadvantages are the waste of fuel and inconvenience consequent on letting out the fire after each charge, and the re-lining, which becomes necessary every twelve months. The tunnel kiln is economical in fuel, requiring only about one-fifth the weight of the lime produced (a flare kiln requiring about one and two-third times this amount), but the lime is not equally burnt, is not so clean, and more experience is required than in the management of a flare kiln.

The Manufacture of Grey Stone Lime.

Grey stone lime is a feebly hydraulic lime obtained from the lower chalk in the south of England, principally at Dorking and Merstham, in Surrey. It is quarried as ordinary stone (calcium carbonate), and burnt in kilns to drive off the carbonic acid, and leave the lime behind in the state of quicklime (calcium oxide). The common flare kiln (Fig. 619) is usually built in a circular form of rough bricks or blocks of stone with a fire hole or draw hole at bottom, and in such a position that the quarry stone can be wheeled and tipped into the top and the lime wheeled away from the bottom, and is often at the foot of a chalk hill, where the labour for haulage will then be a minimum. A rough arch or half arch of the larger pieces of limestone is first built over the fire space, and the remaining space is then filled up with limestone to the top, the spaces between the lumps allowing the heat to pass up readily from the fire, which is generally of wood, and kept burning for three or four days, until the whole of the carbonic acid and moisture are driven off in the form of gas and vapour. Tunnel kilns (Figs. 617 and 618) have the fuel and lime stone in alternate layers, and are worked continuously, but the lime is not equally well burnt throughout.

Testing Qualities of Lime.

The quality would be tested, in the case of *fat lime*, by making it up into plaster and observing whether it remained free from blisters, "blows," or bulges ; in *hydraulic lime* by testing the setting power in still water ; and in the case of *cement*, by making briquettes and testing the tensile strength after seven days. Also, in all cases, by observing the colour, weight, and general appearance. A chemical test would determine whether a sample of lime is mixed or not, if a separate test of each of the different kinds of lime were first made. If a mixture of chalk and lias lime is in the form of powder, microscopic examination would show the mixture, the appearance of each variety of lime having been first determined by testing them separately. The slaking and setting of a sample of the mixture could also be compared with the action of the two varieties treated separately. The chalk lime would give off considerable heat when slaking and would set very slowly ; the lias lime would give off no appreciable heat and would set quickly. A mixture of the limes would show intermediate results between the extremes mentioned above

Slaking Lime.

Slaking is the process of chemical combination of lime with water. When water is added to pure lime (chalk quicklime), the slaking action commences immediately, the lumps swelling up with hissing and crackling; great heat is caused, steam disengaged, and the lime falls to a fine powder with an increase of two to three times its original bulk. On exposure of quicklime to damp air, the same action takes place more slowly. With hydraulic limes the slaking is much slower, the heat less, the increase of bulk less, and the lumps may crack all over, but do not fall to powder. Carbonate of magnesia, sulphate of lime, alkalies, metallic oxides, and clay all modify the slaking action by retarding it, but cause a quicker setting or hardening of the mass after being made into mortar. The hydraulicity of lime mainly depends upon the presence of silicate of alumina, or clay.

Up to 10 per cent. clay, the limes are feebly hydraulic.

Up to 20 per cent. clay, the limes are ordinarily hydraulic.

Up to 30 per cent. clay, the limes are eminently hydraulic.

Varieties of Limes.

Fat lime or rich lime is made from nearly pure carbonate of lime, calcined from the upper chalk, marble, or other beds containing 98 to 100 per cent. carbonate of lime. Slakes fiercely, swelling to two or three times original bulk, with great disengagement of heat and steam, and falls into a bulky powder which, made into mortar, has little setting power. Is very soluble and used chiefly for plastering and whitewashing. Rich lime will carry the most sand, because it is nearly pure carbonate of lime ; the impurities in the others take the place of sand in mixing, and it is necessary to use a larger quantity of sand with the rich lime to prevent excessive shrinking. The setting is very protracted, and only superficial ; colour, creamy white or pale buff. Chalk lime, otherwise known as rich, fat, or pure lime, is used for plastering for the following reasons : It slakes easily and thoroughly. It is economical, the slaking process causing it to swell to two or three times its original bulk. It works easily. It is deficient in strength, but strength is not required. It is porous, and therefore temporarily absorbs the moisture, which condenses on

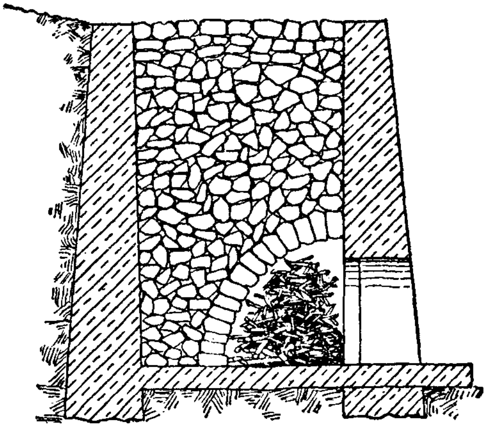


Fig. 619.—Flare Lime-Kiln.

a wall upon a sudden rise of temperature. It is cheap in the first instance. It does not spoil but rather improves by being mixed a considerable time before it is required for use; this is called cooling it.

Hydraulic limes are those made from carbonate of lime containing a mixture of clay, which gives it the power of setting under water—hence the term hydraulic. Hydraulic limes slake almost without heat, and do not fall into powder; they set vigorously even in damp situations, and do not require access of air; colour, pale grey.

Lias lime varies according to the locality it comes from, generally with 20 to 30 per cent. of clay in its composition; it is very difficult to slake, commences after long and uncertain periods, very slight development of heat sensible only to touch, very often no cracking or powder produced. Sets firm under water in twenty hours, is hard in two to four days, very hard in a month, in six months can be worked like a hard limestone and has a similar fracture. The lias lime is used for retaining walls or for foundations in damp soils, is unsuited for plastering, and seldom if ever used for that purpose. The grey lime slakes freely, but is not so economical as chalk lime, and forms a less absorbent surface. Lias lime is very liable to blow when used for plastering, and gives a non-absorbent surface.

Poor lime contains 60 to 90 per cent. of carbonate of lime, together with useless inert impurities, slakes sluggishly and imperfectly, with little increase of bulk; it sets rather firmer than rich limes, owing to the presence of a small quantity of clay, but has no strength.

Quicklime is freshly burnt chalk, or broken limestone, produced in a tunnel kiln or flare kiln. In the former the fuel and stone are placed in alternate layers, and in the latter the fuel is placed below so that only the flame reaches the stone in the kiln above. When water is applied to fresh quicklime produced from soft white chalk it hisses, crackles, and explodes, gets very hot, sucks up the water quickly, produces large volumes of steam, and falls into a bulky powder.

Stone lime, or grey chalk lime, slakes very freely; after being wetted it pauses a few minutes, then slakes with decrepitation, development of heat, cracking and ebullition of vapour. It will set in still water owing to its containing 5 to 12 per cent. of clay, is firm in fifteen to twenty days, and in twelve months is as hard as soap. Stone lime is suitable for building ordinary walls of brick or stone; it is feebly hydraulic, of a light buff colour, and when made into mortar with two parts of sand will sensibly resist the finger nail at a month old.

Origin and Formation of Sand.

Building on sand is proverbially unwise; but to build without it is almost impossible. All natural sand has been produced in virtually the same manner. The igneous rocks, of which the earth's crust was originally composed, were broken up by the action of the weather and other causes, and the fragments were washed down into the valleys, river-beds, and ocean floors, there to form new layers, which in course of time were raised again by the changes produced in the relative levels of land and sea. In the meantime the pressure and heat had produced modifications in the structure of the layers; and as the ancient sea-beds were raised they were subjected to new influences. Metallic veins forced their way from time to time through the overlying rocks, and added their material to the general mass which was worn away by the weather. Organic remains were added, consisting chiefly of carbonates and sulphates of lime, magnesia, etc. The more perishable parts were carried away first—such, for instance, as felspar, producing clay or mud—while the more durable parts, such as the crystals of quartz, came away more or less whole, and by the action of the water were worn down to rounded particles, which ultimately helped to form sandstone. In the West of England the sea sand is white, because it consists of the remains of these quartz crystals, which are nearly pure silica. The chalk rocks are made up of the shelly remains of countless numbers of minute organisms—creatures that once had life—and consist of pure carbonate of lime, the silica which existed in solution in the ancient seas having collected into nodules or lumps in conjunction with the chalk in the form of flints. In the South of England, when the chalk was worn away by wave action, these flints, being exposed, composed the great part of the sand formed there; but in every district the sand contains the remains of larger masses of various kinds, which have been worn down to small grains. There is no part of England that has not been at some time under the sea, perhaps many times, and hence beds of sand and gravel are found far inland. Gravel is only large sand; and when it is dug from a gravel pit, the tawny colour that is observed is due to staining by iron oxides. The essential point to note is that all sand has been formed by attrition — that is, the rubbing down of larger masses by rolling about on the seashore.

Sharp Sand.

What is called sharp sand is therefore only that which, being free from loamy particles or clay, feels hard and rough when rolled in the hand, and does not stain the fingers. Any angularity that occurs in pit sand is due to the accretion of particles by their being cemented together by oxide of iron.

Desirable Qualities in Sand.—It is most important that, for use in mortar or concrete, the sand should be free from clay, as, if any is present, the water forms it into mud, which encases the particles of lime or cement, and prevents them from effecting the cohesion of the mass. In view of this, it will be seen that the finer the cement is ground, the more it is able to resist any accidental admixture of clay. At one time it was considered that Portland cement was ground sufficiently fine when it left 10 per cent. on a sieve of 2,500 holes per square inch, but now it is often specified to be so fine that not more than 12 per cent. shall be left when sifted through a sieve having 10,000 holes per square inch.

Mortar.

Lime mortar in London is usually composed of 1 part of best grey stone lime (Merstham, Halling, or Dorking), and two parts of clean sharp sand. Blue lias lime is a hydraulic lime having the power to set under water, and may therefore be used in wet ground; those named above being suitable only for dry situations. Chalk lime is unsuitable for use in brickwork owing to its solubility and want of setting power. Clean, sharp, pit sand is most suitable for making mortar. Sharp sand is that which has no loam, clay, or other dirt mixed with it, which would retard the setting, by preventing crystallisation. Sea and river sand make good mortar when washed, otherwise the hygroscopic character of the salt would attract moisture and retard or prevent setting. For brick-on-edge coping, the mortar should be composed of 1 Portland cement to 2 or 3 of sand, to hold the bricks firmly and prevent moisture from soaking through to the lower courses. For a tall chimney shaft cement mortar is generally considered too rigid, and a mortar is preferred composed of 1 blue lias lime to 3 sand. For outer wall of warehouse basement, the mortar may be composed either of blue lias lime or Portland cement, as either will be suitable for resisting moisture, and will be of considerable strength. Mortar for flat pointing, 1 of lime or cement to 2 of sand. A struck joint with the upper edge pressed in, and done as the work proceeds, is more durable than flat pointing. Cement pointing is not usually applied to a new wall built with lime mortar, although it is often used in repairs. One cub. yd. of mortar (known as a "load" of mortar) contains 27 cub. ft., and requires, to make it, 9 bushels of grey lime and 1 yd. of sand with a sufficient quantity of water to bring it to a stiff paste. The bushels must be heaped, and will then hold 1½ cub. ft., so that 9 bushels equals 13½ cub. ft., which disappears in bulk when mixed with the 27 cub. ft. of sand. The common proportion for London mixture is 3 to 1, instead of 2 to 1 as given above, but most architects specify the 2 to 1 proportion. A rod of London stock brickwork, laid to the standard of four courses to the foot, requires 71 to 75 cub. ft. of mortar.

Plaster-of-Paris.

Plaster-of-Paris is obtained by grinding gypsum (a soft stone of crystalline texture consisting of hydrated sulphate of lime), and then calcining it in iron vessels until nearly all the water of combination is driven off. It is then in the form of a fine powder like wheat flour in appearance, but heavier. It is used alone or as an addition to other materials. When mixed with water, plaster-of-Paris sucks up a large quantity without perceptible heating and sets hard in a few minutes, expanding in so doing. The explanation of the changes produced is that the calcination drove off the water with other matters in the original material, and left it thirsty (that is, with strong chemical affinity for moisture), and in "slaking its thirst" it retains the moisture which alters both its composition and appearance. Plaster-of-Paris, being very soluble in water, can only be used for interior work.

It is used chiefly for cheap statuary for internal decoration, cornices, centrepieces, and other ceiling ornaments, console brackets, etc., being made into a paste, and forced into gutta-percha moulds. It is also used for filling up holes in walls and ceilings, or other defects, and sometimes in pattern-making for foundry purposes. When mixed with lime for plastering, in the proportion of 1 to 4 or 5, it forms "gauged stuff," and causes the coat to set quicker and harder; but it cannot be used for external work, owing to its solubility and rapid destruction when exposed to the weather. There are three qualities of plaster-of-Paris in the market—"superfine" and "fine," sold in casks of 2 cwt.; and "coarse," sold in sacks or bags from 7 lb. upwards. Weight per striked bushel = 64 lb., weight per cubic foot = 50 lb. The gypsum occurs as selenite in the London clay at Lewisham and elsewhere, and particularly near Paris, in transparent white crystals 4 in. or 5 in. long, 2 in. or 3 in. broad, and 1 in. to 2 in. thick, in irregular masses which break up into cubical pieces. When the gypsum is a translucent or nearly opaque white, it is known as alabaster, and is used chiefly for statuettes and ornaments. When coloured in mixed yellow and brownish shades, it is familiar as Derbyshire spar, used also for vases and ornaments.

Cements.

Cement is an artificial mixture of lime and clay burnt and ground together (except Roman cement, which is a natural product), possessing the properties of hydraulic limes to an eminent degree, and suitable for building in wet situations. Parian and Keene's cement are used for internal plastering and surfaces intended to be painted. Parian cement would be used in preference to Keene's when it is desired to paint the surface as soon as possible, also when the surface to be plastered is large. Keene's cement would be selected for use where hardness was desirable, such as for floors, skirtings, columns, pilasters, etc. Plaster-of-Paris forms the basis of Keene's cement, which is plaster-of-Paris and alum; Parian cement, which is plaster-of-Paris and borax; Scagliola and Marezzo marble. It is the essential element in "solenitic mortar," or Scott's cement, where the addition of 5 or 6 per cent. of plaster-of-Paris to a feebly hydraulic lime checks the slaking and expedites the setting, permitting also a larger quantity of sand to be used without weakening the mortar.

Selenitic Cement.—Selenitic cement, invented by General Scott, R.E., contains a small proportion of sulphate of lime, added in the form of plaster-of-Paris to the carbonate of lime and ground with it. The best lime for the purpose is lias lime, but limes from the magnesian limestone formation and grey chalk are much used. The proportion of sulphate varies from 4 to 7 per cent., according to the nature of the limes. It is generally about 5 per cent., sufficient being added to stop the slaking action and render the setting more rapid and without expansion. The cement should be ground to pass at least a sieve of 900 meshes to the square inch; it allows a much larger proportion of sand to be mixed with it than ordinary lime without loss of strength. Records of experiments show that it makes a stronger mortar than Portland cement, at a less cost. It is also used for plastering, but plasterers say it requires much more time and labour to work up to a good face than common plaster, whilst fire-cracks are liable to develop on the surface. It must not be used where it is required to be gauged with plaster, as for cornices, screeds, etc.

Portland Cement.

Portland cement is an artificial cement so called from a fancied resemblance in its colour to Portland stone. It is by far the most valuable of all the cements, and is made by intimately mixing and calcining together substances of different kinds, so as to obtain a material containing when burnt about one-third part of clay and two-thirds of lime. Materials used are chalk and clay by the wet process, limestone and clay or shale by the dry process. The wet process is usually employed on the Thames and Medway. The colour of the manufactured cement is a bluish grey, but sometimes inclining to a light brown. When well burnt it weighs usually from 110 lb. to 120 lb. per imperial striked bushel. The ordinary variation of weight is from 100 lb. to 115 lb. per imperial striked bushel, but it may be from 95 lb. to 130 lb.; an average for general use may be taken as 112 lb. The weight depends upon the temperature and time of burning, the fineness or coarseness of the grinding, and the mode in which the measure is filled and struck. A striked bushel is a bushel measure gently filled with cement and the surplus swept off by striking a square-edged board across the top. Heavy cements are liable to be overburnt, and require the greatest care in grinding. They should be freed from all coarse unground particles, which add to their weight, and, being very hard and overburnt, slake slower than the remainder, causing the work to "blow." In very heavy cements there is sometimes an excess of lime, rendering the cement unfit for use in sewers or other places where liable to contact with acids. Heavy cements are slow setting, but have a greater ultimate strength than the lighter samples. The heavy cement is used for foundations, retaining walls, and engineering work generally; the light cement for concrete floors, rendering walls, etc. Quality depends upon care in manufacture; under-burning produces greater bulk from a given quantity of material, requires less fuel and grinding, sets more quickly, but never arrives at the same ultimate strength. It sets more slowly than Roman cement, the heavier samples setting slower than the light ones, say one to ten hours. At the end of seven days the tensile strength should be about 1,000 lb. on 1½ in. sq., the strength generally increasing for about two years.

The Manufacture of Portland Cement.

The manufacture of Portland cement is as follows:—The cement best known in this country is made on the banks of the Thames and Medway from chalk and clay mixed by the wet process. The proportion of chalk and clay mixed together depends upon the composition of the chalk before burning. The result required is to obtain a mixture containing some 25 to 30 per cent. of clay. With white chalk (which itself contains no clay) 3 volumes of chalk are mixed with 1 volume of alluvial clay or mud from the lower Thames or Medway. If the chalk itself contains clay, the proportion of clay added is modified accordingly. For example, with grey chalk 4 parts of chalk are used to 1 of clay. The chalk and clay are mixed in water to the condition of a creamy liquid which is called slurry, the fine particles in suspension are allowed to settle in large tanks, reservoirs, or backs for several weeks, and when the deposit becomes nearly solid the water is run off, the residue is dug out, sometimes pugged, dried on iron plates over coking ovens, or over the flues from the kiln, burnt in intermittent kilns at a very high temperature and then ground to a fine powder. Portland cement differs very considerably in its characteristics and action according to its quality. It can be manufactured more cheaply when under-burnt, because then a greater bulk of cement is produced with a given quantity of material, and it requires less fuel and less grinding; it also sets more quickly, but never arrives at the same ultimate strength as a properly burnt cement. Under-burnt cement contains, moreover, an excess of free quicklime, which is apt to slake in the work and cause great mischief. This may be remedied by exposing the cement, and allowing these particles to become air-slaked. The conditions to be fulfilled by good Portland cement depend upon the purpose for which it is required.

Tests for Portland Cement.

A standard specification for Portland cement given in a paper read before the Society of Engineers by Mr. D. B. Butler, cement specialist, is here reproduced. "Pure Portland cement must conform to the following tests: Fineness of grinding: When the cement is sifted through a standard sieve containing fifty holes per lineal inch, there shall not be more than one half (½) per cent. by weight of residue: when sifted through a sieve having seventy-six holes per lineal inch, there shall not be more than five (5) per cent. of residue; and when sifted through a sieve having one hundred holes per lineal inch, there shall not be more than twelve (12) per cent. of residue. Time of set: A pat of neat cement gauged with the minimum of water at the normal temperature (60° Fahr.), and placed on a glass or other non-porous slab, shall not commence to set in less than eight minutes nor take longer than five hours to set hard. Soundness, or freedom from expansion and contraction: A pat submitted to moist heat and warm water in the Faija apparatus for soundness at the usual temperatures, namely, 110° Fahr. and 120° Fahr. respectively, shall show no cracks or signs of expansion after twenty-four hours. Tensile strength: Briquettes of neat cement, gauged with the minimum of water on a non-porous bed, and placed in water twenty-four hours after gauging, shall carry an average tensile strain of not less than 350 lb. per square inch after three days, 450 lb. after seven days, and 550 lb. after twenty-eight days from the time of gauging. Briquettes composed of three parts of standard sand to one part of cement, by weight, treated as above, shall carry an average tensile strain of not less than 450 lb. per square inch at seven days, and 550 lb. at twenty-eight days from the time of gauging; but no matter how much greater strength may be developed at the earlier dates, both neat and sand briquettes must develop an increase of at least 50 lb. between each date." The fineness of grinding is to ensure particularly that the over-burnt hard particles shall be properly broken up, otherwise they will slake after the work is completed, and may do great damage. The time of setting is important; if the cement sets too quickly, it is not likely to have a great ultimate strength, whereas if the cement sets too slowly it may be stale. The hot-water test shows whether the cement contains an excess of clay, which would reduce the strength. The bottle test shows whether the cement has been properly air slaked in order to reduce the effect of excess of lime. The test for tensile strength is the one generally most relied on, as giving an absolute guarantee of quality. The colour test shows by a brownish tint a probability of under-burnt cement; if blackish, an excess of over-burnt particles; if bluish grey, a probability of excess of lime. The proper colour of good cement is a grey with a slight greenish blue tint.

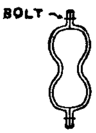


Fig. 620.—Mould for Briquettes.

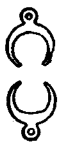


Fig. 621.—Clips for Cement Briquette.

Testing Portland Cement for Tensile Strength.

The testing of Portland cement for tensile strength, to be of any practical use, requires a testing machine, which, with accessories, in a small way cannot be purchased for less than £25. The operator then requires a course of training before he is sufficiently expert for results to be reliable. For amusement, a testing machine may be made as follows: Procure some brass moulds as Fig. 620, filed quite smooth, ½ in. deep and ½ in. wide at the narrowest part, in which to make up the briquettes while laid on a sheet of plate glass; a pair of brass clips as Fig. 621, in which to hold the briquettes for testing; and a trestle with suspending hook and scale pan (Fig. 622) for carrying the weights. The chains holding the scale pan should be stout, as the weight required will be about 125 lb. A folded sack may be placed on the floor under the scale pan to catch it when it drops on breakage of the specimen. The Fairbanks Company of New York, and of 16, Great Eastern Street, London, E.C., make a small and compact testing machine at a cost of about £17, and supply all the other apparatus necessary for a complete testing station. The inclusive cost of a small set would be from £50 to £60. George Salter & Co., of West Bromwich, supply a small machine and accessories suitable for a class-room demonstration for about £30.

Testing Portland Cement on the Building Site.

Owing to the refinement of modern specifications, testing cement is now a matter that must be left to the expert; a clerk of works cannot hope to do more than make a very rough test. Testing for heat may be done by thrusting the bare arm into the fresh dry cement, which should feel cool, but not cold or dead. When hot the cement is likely to be under-burnt,

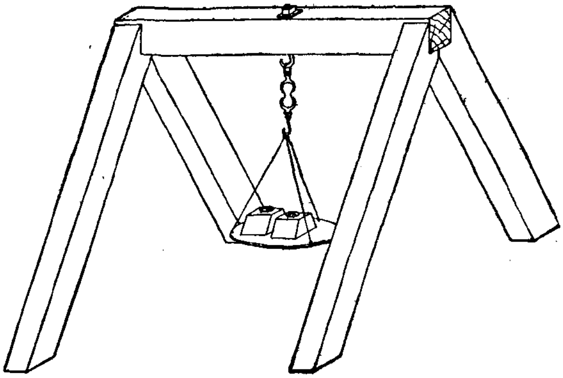


Fig. 622.—Cement Testing Device.

and to contain an excess of lime, and must be cooled by air slaking. If the sample of cement to be tested is gauged stiffly and filled into a thin glass test tube, the tube will crack while the cement is setting if it has not been sufficiently air slaked; and if under-burnt the cement may shrink and become loose. A pat of cement about 3 in. in diameter, ½ in. thick in the centre, and with thin edges, made up on a piece of slate and put into cold water for twenty-four hours, should show no signs of cracking round the edges. Boiling such a pat is a much more severe test, and is sometimes specified.

Roman Cement.

Roman cement, originally called Parker's cement, is a natural cement obtained by calcining nodules found in the London clay It consists approximately of clay 50 per cent., lime 40 per cent., oxide of iron, etc., 10 per cent. The colour is generally a rich brown. Good Roman cement should weigh not more than 70 lb. to 75 lb. per trade bushel, and should set quickly, say in a quarter of an hour. It does not attain any great strength, and is much weakened by admixture of sand. At the end of seven days the tensile strength of neat cement is about 150 lb. on 1½ in. square, but with equal parts of sand and cement the strength would be only about one-third of that amount.

Medina Cement.

Medina cement is made from the septaria found in Hampshire and the Isle of Wight, and from those dredged up out of the bed of the Solent. It sets very rapidly, is of a light brown colour, and resembles Roman cement in its characteristics; but is stronger. Septaria are flattened masses of clayey matter showing crystalline seams.

Robinson's Fireproof Cement.

Robinson's fireproof cement has been used as a substitute for Keene's and Parian cement. According to Mr. Faija, the authority on cements, "It is slow setting, easy to work and manipulate, and may be used for an hour and a half after being gauged without detriment to its ultimate strength. It attains after a very few hours great strength and hardness, which greatly increase with age. In setting, it neither expands nor shrinks." It is made in three qualities, No. 1 for finishing coat on walls and ceilings, No. 2 for first coat in plastering, No. 3 for concrete. It will carry a very large proportion of sand, but 2 to 1 for ceilings and 3 to 1 for walls and partitions may be generally recommended. At 3 to 1, 1 cwt. of cement will cover 15 yd. super. ½ in. thick. The maker's instructions should be followed in using it. The quality of sand used with the backing coat is of great importance. If good coarse clean sand is used the proportion may be 2 to 1 of cement for ceilings, angles, lath-work, and mouldings, and 3 to 1 for plain walls; but if the sand is fine or loamy, or of an inferior quality, one part less (at least) should be used.

Plasterers' Materials.

The basis of ordinary plaster is calcium carbonate, and of the hard coats calcium sulphate. The calcium carbonate or lime is generally pure or fat lime from the upper chalk, calcined and thoroughly slaked. The calcium sulphate or plaster-of-Paris is produced by the gentle calcination of gypsum to a point just short of the total expulsion of moisture.

Coarse Stuff is a rough mortar containing 1 to 1½ parts of sand to 1 of slaked lime by measure, and 1 lb. ox-hair to every 2 cub. ft. to 3 cub. ft. of stuff. The sand, which should be clean and sharp, that is, free from clay or earthy impurities, is first heaped round in a circular dish form on hard level ground. The lime, previously slaked and mixed with water in a large wooden tub to a creamy consistency, is then poured into the middle. The hair—long, sound ox-hair from the tanner's yard, free from grease or dirt, and previously well switched with a lath or immersed in water to separate the hairs—is then added, and well worked in throughout the mass with a three-pronged rake. The mixture is then left for several weeks to cool, that is, to become thoroughly slaked to prevent blowing after being laid.

Fine Stuff is pure lime slaked to paste and afterwards diluted to the consistence of cream. It is then allowed to settle, the water rising to the top is run off, and the stuff is left till it is thick enough for use. A little white hair is added for some purposes.

Plasterer's Putty is pure lime slaked with water, brought to a creamy consistence, strained through a hair sieve, and allowed to evaporate until stiff enough for use. It is the last coat applied to internal walls that are to be coloured, and always is used without hair.

Gauged Stuff is plasterer's putty with a portion of plaster-of-Paris mixed with it, the proportions being 3 parts putty to 1 part plaster-of-Paris when required to set quickly, and gauged in small quantities.

Laths: Sawn and Split.

General opinion is undoubtedly in favour of split laths, and split laths are always specified by architects for ceilings and partitions. Sawn laths, unless cut from specially selected straight-grained stuff, would most assuredly have weak places from uneven grain, and in order to avoid this weakness the sawn laths would have to be made thicker than split laths. "Specification" says, "Baltic fir is used for laths for modern work; only the best quality should be used. Oak laths, formerly used, are very liable to warp. The defects that are to be avoided in laths are sap, knots, crookedness, and undue smoothness. The sap decays; the knots weaken the laths; the crookedness interferes with the even laying on of the stuff, and the undue smoothness does not give sufficient hold for the plaster on the lath. Riven laths, split from the log along its fibres, are stronger than sawn laths, as in the latter process the fibres of the wood are often cut through. Sawn laths are, however, cheaper than riven laths, and are superseding them, which is not desirable in good work. Laths are sold in three sizes, namely, 'single' (average ⅛ in. to $\frac{3}{16}$ in. thick), 'lath and half' (average ¼ in. thick), and 'double' (⅜ in. to ½ in. thick). The thicker laths, because of the strain upon them, should be used in the ceilings, and the thinner laths should be used in vertical partitions, etc., where the strain is but small. Some walls and partitions have to stand rough usage; in such cases the thicker laths are necessary. Laths are usually spaced with about ⅜ in. between them for key. A bundle of laths usually contains 360 lin. ft., and such a bundle, nailed with butt joints, covers about 4½ super. yd., and requires about 500 nails if the laths are nailed to joists 1 ft. from centre to centre. The lengths of laths vary from 2 ft. to 5 ft., but 3 ft. to 4 ft. are the usual lengths. Laths are best nailed so as to break joint entirely, because, for various reasons, there is a tendency to crack along the line of the joints if the laths are nailed with the butt ends in a row. This may be obviated by using 3 ft. and 4 ft. laths together; ceilings are much stronger if the laths are nailed in this way. Laths, however, are usually nailed in bays, about 4 ft. or 5 ft. deep. Every lath should be nailed at each end, and also at the place where the lath crosses a joist or stud. Lap joints at the ends of laths, which are often made in order to save nails, should not be allowed, as this leaves only ¼ in. for the thickness of plaster. Butt joints should always be made. Joists, etc., that are thicker than 2 in., should have small fillets nailed to the under side, or be counter lathed, so that the timber surface of attachment may be reduced to a minimum and

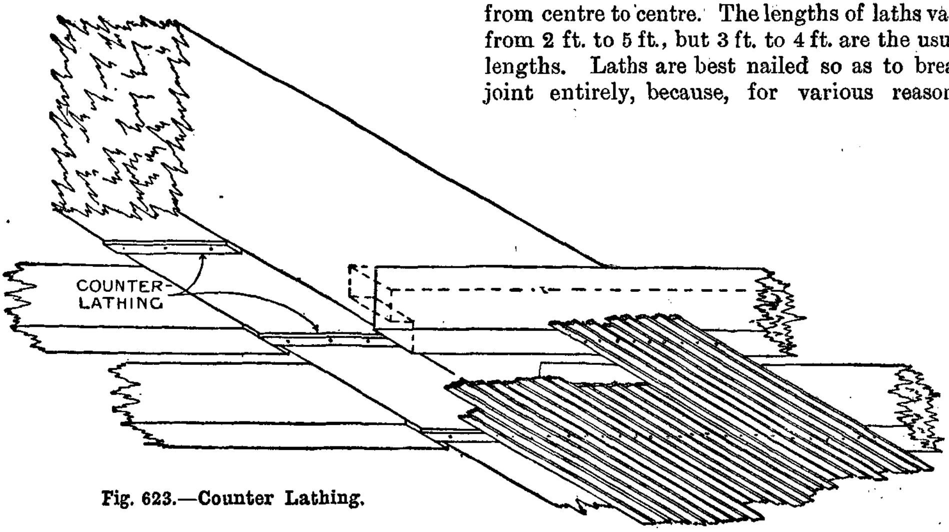


Fig. 623.—Counter Lathing.

the key not interfered with. Lathing nails are usually of iron, and are galvanised, cut, wrought, or cast; where oak laths are used, the nails should be galvanised or wrought. Galvanised nails should also be used with white cement work. Zinc nails, which are expensive, are used in very good work, because of the possibility of the discoloration of the plaster by the rusting of iron nails. The length of lathing nails depends on the thickness of the laths, ¾-in. nails being used for single laths, 1-in. nails for lath-and-half laths, and 1¼-in. nails for double laths."

Counter Lathing.

Counter lathing (Fig. 623) consists in nailing short pieces of laths at 12 in. intervals across a beam or similar surface which comes in the way, so that lathing and plastering may be continued across it in the contrary direction without interrupting the key.

Plastering a Plane Surface.

The battens should be tested, before the lathing is done, to see that they are in one vertical plane, and trued up if necessary. The lathing should be best Baltic fir, lath-and-a-half laths, say 1 in. by ¼ in. by 4 ft. The laths should be laid side by side about ⅜ in. apart, with butt ends, one nail at each end, and one at each crossing of a stud. They may be snatched (or matched)—that is, with the joints in line for 2 ft. or 3 ft. on walls and 18 in. on ceilings. Where any timber over 3 in. wide is crossed, a narrow fillet or double lath should be laid lengthwise, called counter lathing, to leave sufficient key for the plaster between the laths and the timber. The first coat of "coarse stuff," or "pricking-up" coat, is a rough mortar made as before described. It should be worked up well with the trowel, and though thin enough to key up, or pass readily between the laths, it should be stiff enough not to break away from the keying as it is put up. The trowel should be worked across the laths at an angle of about 45°. The thickness from the face of the laths should be about ⅜ in. After this coat is put on, while still soft, it is scratched or scored all over with the end of a lath, which should be held at an angle to the face of the work, in order, by undercutting the scoring, to form a better key for the floating coat. The scoring should be carefully done in parallel lines about 3 in. apart, and then crossed over with a second set of parallel lines at an angle to the first. A special scoring tool, with several teeth, is sometimes used to expedite the process. Plasterers often take several laths in their hands at the same time and scratch the surface over at random, but this should not be allowed.

Floating and Setting a Plane Surface.

When the pricking-up coat is quite firm but not too dry, the second or floating coat, consisting of either a repetition of the coarse stuff or of fine stuff containing a small proportion of hair, is put on. The operation of floating is performed by first surrounding and dividing the surface to be floated, with screeds or gauges for level and thickness, formed at top and bottom by the cornice grounds and skirting grounds, and at intermediate points by vertical lines of plaster called wall screeds. The bays are then filled up and brought to a level surface by a floating rule or long straightedge called a Derby float. Before the floating coat is too dry it is swept over with a birch broom, and when the floating coat is quite dry the third or setting coat is applied, which, if to be painted, should be of bastard stucco trowelled, composed of two-thirds fine stuff, one-third very fine clean sand, and with or without a little hair. This is set with the largest trowel, brought to a smooth face over a surface of 2 yd. or 3 yd., and then worked over with the hand float, at the same time wetting the surface with a brush and floating and sprinkling it alternately (called "scouring") until it presents a hard polished appearance, after which it is rubbed over with a dry stock brush. To prevent this coat from being disfigured by fire-cracks, it must not be applied until the previous coat is perfectly dry.

Screeds and Screeding.

Screeds in plastering are strips of plaster (second coat) 6 in. or 7 in. wide and 4 ft. to 10 ft. apart, carefully levelled and plumbed, as a guide for running the float over the remainder of the work. In cement rendering they occur in the first or only coat. Sometimes the screeds are narrow battens of wrought deal, used for the same purpose as the plaster screeds. The cement reveals to window openings are sometimes called screeds. The bedding of window frames in mortar to prevent draughts is sometimes called "screeding them in."

Trowelled Stucco.

Trowelled stucco is used for finishing internal surfaces when they are intended to be painted. It is composed of two-thirds fine stuff, without hair, and one-third very fine clean sand.

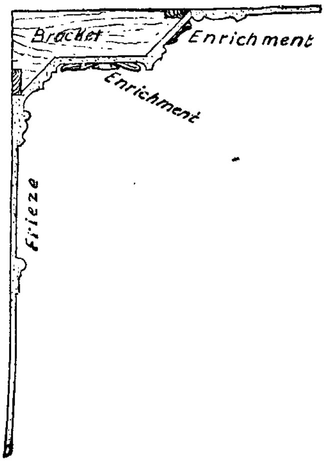


Fig. 624.—Section of Bracketed Cornice.

Plastering Drawing-room.

Assume the case of a drawing-room 30 ft. by 25 ft., ceiling 14 ft. from floor, which is to be plastered complete. The two 25-ft. walls and one 30-ft. wall are outside walls. There are two doors, one large double-leaved door and one ordinary-sized door (in a good house); there are four windows. The combined area of doors, windows, and fireplace over the level of skirting grounds is 360 sq. ft. Skirting grounds are 1 ft. from floor. If the three outside walls are to be battened for lath and plaster, they should be plugged for battens and for the skirting and cornice grounds, and the battens and ceiling joists should be tested for level and plumb before the lathing is commenced, so that the plastering may be of uniform thickness. The other wall may be raked out and rendered, if brick, or treated the same as the external walls. If wooden or bricknogged partitions, they would be properly lathed and counter lathed. The cornice would be bracketed, and proper angle brackets should be provided. The lathing should be laid with butt joints properly snatched. The first coat of coarse stuff, consisting of 1½ part sharp sand, 1 slaked lime, with 1 lb. of clean, sound, long ox-hair to every 3 cub. ft. for the walls, and every 2 cub. ft. for the ceilings, is to be laid on with a good key. When drying, it is to be well scratched or scored to form a key for the second coat of similar composition. After the second coat is laid the cornice is to be roughed out with gauged coarse stuff and then run with a mould in gauged putty. The enrichments are then to be prepared and fixed with gauged putty. The cornice is to be as in Fig. 624. For quantities say

Ceiling 26 × 21 = 546 ft. sup.
Walls 110 × 12 − 360 = 960 „
1506

	£	s.	d.
$\frac{1506}{9}$=168 yd. @ 2s. 9d.	23	2	0
Cornice and frieze say 110 ft. by 5 ft. girth = 550 sq. ft. @ 9d.	20	12	6
Enrichments say 200 ft. run @ 2s.	20	0	0
	£63	14	6

Running Cornice round Room.

It is assumed that a cornice of the cross section shown in Fig. 625 is to be formed round the ceiling of a room. The cornice will require to be bracketed—that is, to have pieces of wood cut to shape to receive the laths upon which the cornice will be built. The brackets are fixed against proper grounds, and lathed before the first coat of plaster is put upon the walls and ceiling, so that all the plasterer's work may be gone on with consecutively. Screeds having been formed upon the wall and ceiling, and running rules fixed to guide the

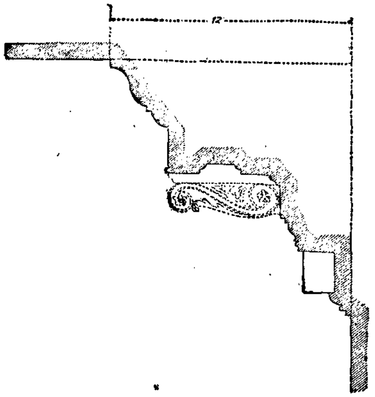


Fig. 625.—Section of Bracketed Cornice.

running mould, the latter is muffled, and the cornice roughed out with gauged coarse stuff. The true mould is then substituted for the muffle plate, and plaster-of-Paris mixed with lime putty used for forming the finished cornice so far as the straight moulded work is concerned. The dentils are formed by casting or moulding between two boards with saw cuts in which are inserted plates of clean sheet zinc to form a series of cells. They are fixed by covering the abutting surface with thin gauged putty and pressing them into place.

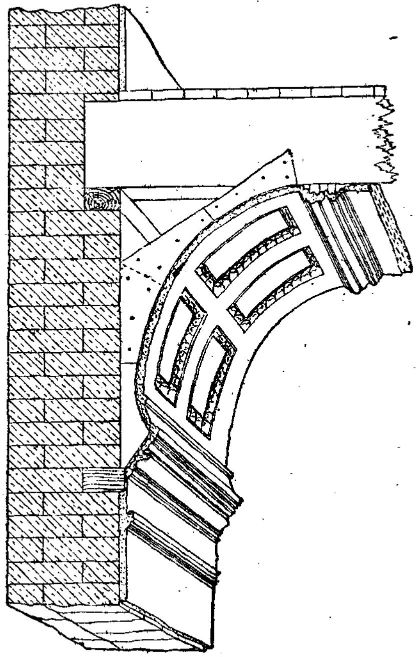


Fig. 626.—Coved and Panelled Cornice on Expanded Metal Backing.

Mitres are formed by continuing the lines formed by the mould into the angle with a straightedge and working up the junction with small pointed mitring tools of various shapes. Fig. 626 shows a coved and panelled cornice for a plastered ceiling laid on expanded metal backing with wooden brackets, the height being 3 ft. and the projection 20 in.

Specification for Plasterer's Work in a First-class Dwelling House.

The sand for plastering is to be freshwater, river or pit sand, and free from earthy, loamy, or saline material, to be well screened, and to be washed if required. The laths to be straight-riven Dantzic heart-of-fir laths of the strength known as lath-and-half, well nailed with 1-in. galvanised lath nails, properly spaced for key, and with butt-headed joints, double-nailed, and breaking joint in 3-ft. widths. The lime for coarse stuff to be approved well-burnt grey stone lime, to be run at least one month before being required for use, to be kept clean, and well mixed as required with 2 parts sand and 1 part lime. The coarse stuff for ceilings, lath partitions, and elsewhere, where directed, to have 1 lb. of good long clean cow-hair, free from grease, loading, or other impurities, well beaten in and incorporated with every 3 cub. ft. of coarse stuff. Approved chalk lime, free from lumps, flares, or core, is to be used for setting, putty, etc., and is to be run at least one month before being required for use. The Portland cement is to be of the best quality and description for plastering purposes, from an approved manufacturer, and must on no account be used fresh, but spread out to cool for at least two weeks in a dry shed or room. All Keene's

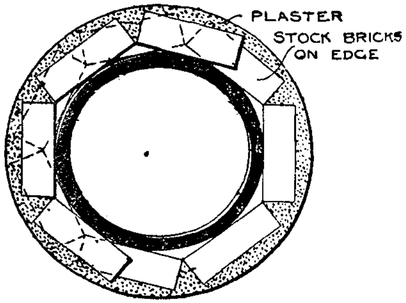


Fig. 627.—Iron Column protected with Bricks and Plaster.

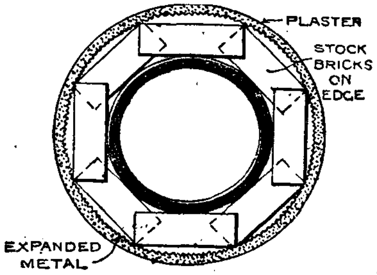


Fig. 628.—Iron Column protected with Bricks, Expanded Metal, and Plaster.

cement and all other materials required in plastering are to be of the best of their respective kinds and descriptions. Provide all plasterer's plant, necessary scaffoldings, tools, moulds, running rules, straightedges, templates, etc., of every kind and description necessary for the proper execution of the work. Lath, plaster, float, and set all wood joist ceilings and soffits, and the quartered partitions in reception rooms. The concrete ceilings and soffits are to be well hacked for key and floated and set in gauged stuff, and the concrete partitions are to be floated and set. Do all dubbing out where required to concrete ceilings, soffits, and partitions in gauged stuff. Render, float, and set all walls where not otherwise described. The walls of entrance hall and staircase to be finished in trowelled stucco for painting. All cornices and moulded work throughout to be run clean and accurately to the sections given. All mitres and returns to be truly worked, and all enrichments and modelling to be to architect's approval, and strictly in accordance with the models and instructions given. Run moulded plaster cornices 18-in. girt with 6-in. modelled enrichment to reception rooms with all mitres returns stopped and mitred ends, etc., as required. The cornices to hall and staircase are to be 12-in. girt run in fibrous plaster, fitted and fixed with proper galvanised nails, and made good to. Provide and fix No. 2 modelled console brackets as detail to junction of hall and staircase. All narrow reveals, splays, and returns to be finished in Keene's cement on a Portland cement backing. Run Keene's cement angles and arrises on Portland cement backing to all projecting angles. Run all necessary quirks, splays, arrises, etc., and make good to all mantelpieces. Cut away for and make good after all other trades, and cut out and make good all cracks, blisters, and other defects, and leave plaster work perfect at completion. Twice distemper, white, all ceilings, soffits, and cornices.

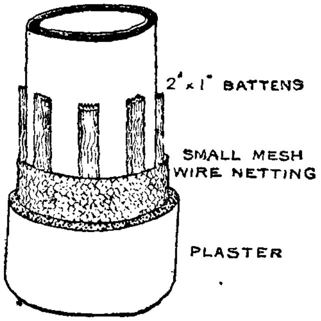


Fig. 629.—Iron Column protected with Wire Netting and Plaster.

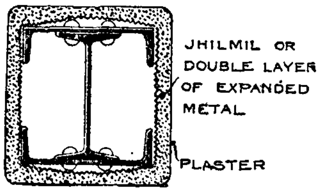


Fig. 630.—Steel Stanchion covered with Jhilmil, or Expanded Metal, and Plaster.

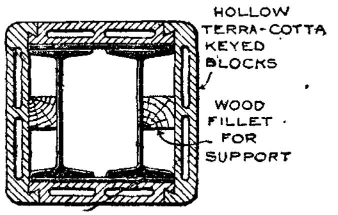


Fig. 631.—Steel Stanchion protected with Terra-cotta Blocks.

Securing Plaster to Iron Columns.

Various methods of protecting iron columns with plaster are illustrated by Figs. 627 to 632. An air space as a non-conductor between the column and the covering is desirable, but this space is sometimes objected to on account of the room occupied. Fig. 627 shows a cast-iron hollow column protected by stock bricks laid close together, but without cutting, and breaking joint in alternate courses. The plaster is then rendered over all and brought to a finished surface. Robinson's fireproof cement may be used for this purpose. Fig. 628 shows bricks placed just far enough apart to support the bricks in the next course, and surrounded by

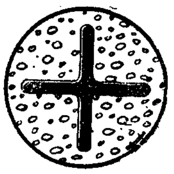


Fig. 632.—Cast-iron Stanchion with Wrought-iron Casing and Concrete Filling.

sheets of expanded metal upon which the plaster may be laid. Fig. 629 shows small slate battens or other vertical strips of wood surrounded by a small-mesh wire netting covered with plaster. Fig. 630 shows a built-up steel stanchion covered with jhilmil, or two layers of expanded metal, and finished with plaster. Fig. 631 shows an arrangement of hollow terra-cotta blocks round a built-up steel stanchion. Fig 632 shows a cast-iron stanchion with wrought-iron casing and fine concrete filled in between.

Blistering of Plaster.

Blistering is a defect to which internal plastering is subject. It takes the form of small patches swelling out beyond the plane of the plastered surface, and is due to the slaking of particles of the lime after the plaster has been applied. To prevent it the stuff should be made a long time before it is required, and left for some weeks to cool. In a very bad case of blistering, the lime used probably contained large hard overburnt particles, which, slaking afterwards, expanded and caused bulges. Sifting the lime before using would have removed these particles.

Efflorescence on Plaster.

On a newly plastered wall an efflorescence like white dust is generally said to be caused by salt in the sand or bricks, and will pass off with ventilation and brushing down the wall. Efflorescence often occurs on new brickwork, but seldom on plaster ; on old work efflorescence occurs on both brickwork and plaster when the place is damp and ill ventilated. Salt is a deliquescent substance—that is, it attracts moisture from the air ; and hence, in rainy or damp weather, the salt that is used at table is always found to be damp. A very small quantity of salt in sea sand may be detected by putting the sand in water and adding a few drops of nitrate of silver ; the presence of salt is shown by a milkiness that is produced by the formation of chloride of silver.

Rendering in Cement.

"Rendering" is the term used for the process of applying plaster or cement to the naked surface of walls. The surface of the wall to be rendered should be rough, so as to form a key to which the plaster will firmly adhere This may be secured by leaving the mortar joints unstruck and protruding when the wall is built; or the joints may be raked and the face hacked and picked over to give it the necessary roughness. Rough rendering is the first coat laid to receive more finished work. It is of coarse stuff, but contains a little less hair than that used on laths, and is applied in a moister state, which causes it to adhere better to the wall. The holes and crevices in the wall should be entirely filled up in applying this coat, but the surface of the plaster need not be scratched or scored over. Floating and setting are performed exactly in the same way as upon laths. For rendering in cement, the wall itself should be dry, but the surface should be well wetted, to prevent it from absorbing at once all the water in the cement; it should also be sufficiently rough to form a good key for the cement. Screeds may be formed on the surface, and the cement should, if possible, be filled out the full thickness in one coat, and of uniform substance throughout. Any excess of cement in projections, mouldings, etc., should be avoided by dubbing out with bits of brick. The purpose served by rendering depends upon circumstances. On a chimney breast or chimney stack it helps to prevent the passage of fire and smoke through any open joints if the pargetting should get damaged. On the interior of a stone wall it prevents osmotic action, or the deposition of moisture upon changes of temperature. In such a position and on the inside of a brick wall it presents a smooth surface, which may be marked with lines to represent ashlar work, or distempered, papered, or painted. As an external plinth, rendering in cement prevents damp reaching the brickwork, and improves the appearance. Many other purposes may be served by rendering.

Rendering Damp Walls in Cement.—Rendering is commonly done to improve the appearance or to prevent damp from reaching a wall, such as sea spray or rain. If the wall is liable to damp, it should be as dry as possible before the cement is put on, or the cement will perish. If the wall absorbs moisture from the soil it is almost impossible to get the cement to stand. For the first coat the mixture should be 2 of pit sand to 1 of cement, and for the finishing coat 1 to 1 ; too much cement in the mixture makes it technically too rich, and is likely to cause cracks.

For mouldings and smooth faces the sand should be carefully fine-screened. The first coat should have set quite hard by the second day; it may then be well wetted and the second coat put on. The last coat should not be touched with water, and if rain gets to it before it is quite dry it is liable to perish; on the other hand, fine cracks often arise through the under-coat being too dry when the last is put on. For rendering inside parapet walls, cement and sand are used without gravel, the mixture being from 5 sand and 1 cement to 3 sand and 1 cement.

Covering Power of Cement.—It may be stated that 1 bushel of neat cement covers an area of 2·8 yd., or $2\frac{3}{4}$ yd. super. $\frac{1}{2}$ in. thick. This statement is worked out as follows:—There are 21 bushels in a cubic yard of 27 cub. ft., therefore 1 bushel $= \frac{27}{21}$ cub. ft. There are 1,728 cub. in. in a cubic foot, and therefore $\frac{27 \times 1728}{21}$ cub. in. in a bushel. There are 9 sq. ft. in 1 sq. yd., and 144 sq. in. in 1 sq. ft., therefore there are 9 by 144 sq. in. in a square yard. Then, multiplying by 2, the cubic inches in a bushel give the number of square inches $\frac{1}{2}$ in. thick, which is the thickness required for the rendering. So that we have $\frac{27 \times 1728 \times 2}{21}$ to divide by 9×144. This will be $\frac{27 \times 1728 \times 2}{21 \times 9 \times 144} =$ say 3·43. But before rendering is applied, the joints have to be raked out, and will require some of the cement to fill them. Allow 18 per cent. for this, then $\frac{18}{100}$ of $3{\cdot}43 = {\cdot}62$, and $3{\cdot}43 - {\cdot}62 = 2{\cdot}81$, or say $2\frac{3}{4}$ sq. yd. covered by 1 bushel of neat cement $\frac{1}{2}$ in. thick.

Roughcast.

Roughcast consists of washed sand, grit, or gravel mixed with hot hydraulic lime (such as blue lias) in a semi-fluid state; it is used as a cheap protection for external walls. The surface of the wall is first "pricked up" with a layer of "coarse stuff," upon which a coat of similar composition is evenly spread; while this is wet, and as fast as it is done in small portions, roughcast in a semi-fluid state is thrown upon it with large trowels from buckets, forming a rough adhering crust, which is at once coloured with limewash and ochre. Depeter consists of a pricked-up coat with small stones pressed in while it is soft, so as to produce a rough surface. Scaling off some time after execution of the work may be due to the under coat being too dry when the outer coat was put on, or to the lime in the roughcast being insufficiently slaked, as may easily happen if blue lias lime be used.

Pricing Plasterers' Work.

For 10 yd. super. of lath, plaster, float, and set—

		s.	d.
5 bushels chalk lime ...	@ 9d. bush.	3	9
5 bushels sand	@ 5d. „	2	1
3¼ lb. hair	@ 1d. lb.	0	3¼
27 gals. water	@ ½d. gal.	1	1½
2¼ bdls. laths and nails ...	@ 2s. 8d.	6	0
Labour, 10 yd. super. ...	@ 6d.	5	0
		18	2¾

With 25% (5s.) added as profit = 2s. 4d. per yard.

Ten yards super. of render, float, and set require—

		s.	d.
5 bushels chalk lime ...	@ 9d. bush.	3	9
5 bushels sand	@ 5d. „	2	1
3 lb. hair	@ 1d. lb.	0	3
25 gals. water	@ ½d. gal.	1	0½
Labour, 10 yd. super. ...	... @ 4d.	3	4
		10	5½
	Add profit 25% ...	2	6
	= 1s. 4d. per yard.	13	0

CARPENTRY AND JOINERY.

Carpenters' Work Distinguished from Joiners' Work.

Carpenters' work is mostly constructional, being done away from the bench, out of doors, or on the carcases of buildings. The timber is used in the rough, and in comparison with its size and value the labour expended upon it is very small. In the "Quantities" allowance is made for tenons and other pieces hidden, overlapping, or necessarily cut off, and the labour is sometimes billed separately from the material.

Joiners' work is of a lighter and more highly skilled character, dealing chiefly with internal fittings and finishings prepared at the bench, the material being always sawn and the exposed parts planed. In the "Quantities" all measurements are net dimensions of the finished work. The labour generally exceeds the value of the material, and is always included with it.

There are some exceptions—for example, wrought timber roof, billed with carpenter but frequently prepared in joiner's shop. Floors billed with joiner but usually prepared in the mill and fixed by carpenters.

Safe Load on Wood Beam.

To find the safe load in cwt. uniformly distributed along a beam, multiply the breadth in inches by the depth in inches, and again by the depth in inches, and then divide by the length in feet. This allows a factor of safety of seven on fir. If the load is applied suddenly the beam will carry only half the above weight, the same as if the load were applied in the centre; but if the load is central and suddenly applied, allow one-fourth only.

Strengthening Timber Beams.

Timber beams are strengthened with iron flitches; for instance, a timber beam, 12 in. by 14 in., can have a $\frac{3}{4}$-in. iron flitch, as in Fig. 633, which shows sectional and elevational views. The arrangement of the bolts is made quite clear. The timber beams alone could support a load of 118 cwt. over a span of 20 ft.; if strengthened as illustrated, it could support a load of 164 cwt. over that span.

Two Wooden Beams Compared.

In comparing two wooden beams, one 3 in. by 9 in., and the other 5 in. by 7½ in., both as regards their resistance to breaking and bending, it will be assumed that in variety, quality, length and manner of loading and supporting, both the cases are identical. Formula for strength, $W = \frac{c\,b\,d^2}{L}$; formula for stiffness, $S = \frac{c\,b\,d^3}{L^3}$.

The comparisons are therefore, for strength, $3 \times 9^2 = 243$; $5 \times 7\frac{1}{2}^2 = 281{\cdot}25$; and for stiffness $3 \times 9^3 = 2187$, and $5 \times 7{\cdot}5^3 = 2109$. Ratio for strength, $\frac{243}{281{\cdot}25} = \frac{1}{1{\cdot}16}$; the ratio for stiffness, $\frac{2187}{2109} = \frac{1{\cdot}037}{1}$. So that the 3 by 9 beam is the stiffer and the 5 by 7½ beam is the stronger.

Pitchpine Beam and Rolled Joist Compared.

A pitchpine beam is strong when it is new, but becomes brittle with age, and should not, therefore, carry a heavier load than a similar beam of Baltic fir. The safe load on a 12 in. by 12 in. beam of 20 ft. span would therefore be $\frac{b\,d^2}{L} = \frac{12 \times 12^2}{20} = 86{\cdot}4$ cwt., say 4⅓ tons. A rolled steel joist for a 20-ft. span should not, in order to avoid excessive deflection, be less than 10 in. deep. A 7-in. by 7-in. by 40-lb. rolled steel joist should only be loaded with about 2½ tons distributed load on a span of 20 ft.

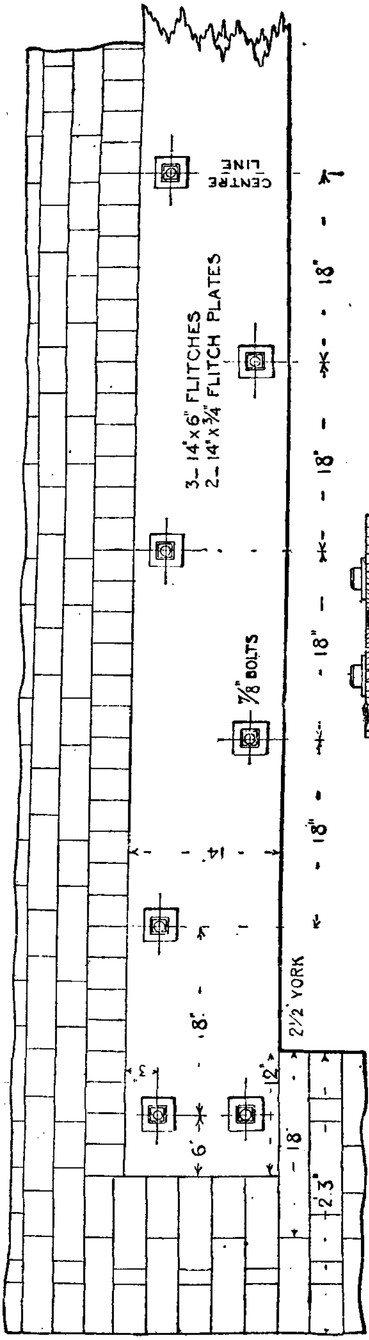


Fig. 634.—Flitched Bressummer.

An 8-in. by 6-in. by 35-lb. rolled steel joist of 20-ft. span would carry 5 tons, while a 10-in. by 4½-in. by 30-lb. rolled steel joist would carry 6 tons, and would cost less than the other joists owing to the lighter weight.

Relative Cost of Rolled Joist and Pitchpine Beam.

Taking a rolled steel joist 6 in. by 3 in. by 16 lb. per ft. run over a clear span of 12 ft., and having a total length of 13 ft., the weight would be 208 lb., and the cost delivered about 15s. to 18s. This joist would carry on the 12-ft. span a safe load of 3 tons distributed with a factor of safety of 4. Pitchpine, although a strong wood when fresh, is apt to get brittle with age, and a suitable size to carry 3 tons distributed over 12-ft. span would be not less than 10 in. by 8 in. A piece of pitchpine sawn all round to 10 in. by 8 in., and 13 ft. long, would contain 7¼ cub. ft. The cost of pitchpine fluctuates considerably, but a mean price delivered would be about 2s. 6d. per ft. cube the cost of the beam being thus 18s. 1½d., so that practically the rolled joist and the wood beam would cost about the same amount.

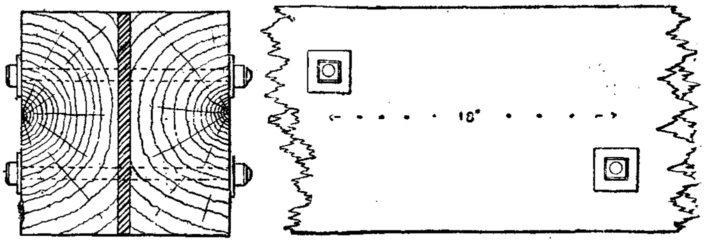


Fig. 633.—Flitched Beam in Section and Elevation.

Determining Size of Girder.

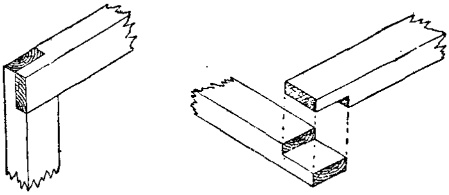
Case 1.—It is required to know the dimensions of a wood girder for a shop front, the opening being 14 ft. The storey above will be of brick, 9-in., and 9 ft. 6 in. high, in which are two openings, each 5 ft. 6 in. by 3 ft. It is assumed that the wood girder or bressummer would have to carry the first floor as well as part of the roof, so that the actual load on it might reach 15 tons; the probability, however, is that it would not exceed 10 tons. The safe load distributed on a bressummer consisting of three 11 × 3 planks over a 14-ft. span would be only $\frac{b\,d^2}{L} = \frac{9 \times 11^2}{14} = 78$ cwt. It would be necessary to insert two wrought-iron flitch-plates, each ½ in. thick, in the bressummer. The strength would then be

$$W = \frac{d^2}{L}(c\ b + 30t) = \frac{11^2}{14}(3 \times 9 + 30 \times 1$$

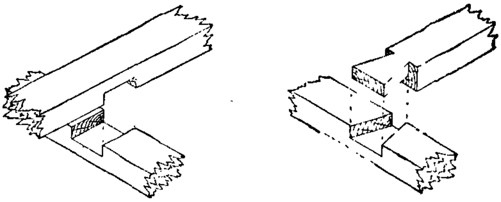
$$= \frac{121}{14}(27 + 30) = \frac{121 \times 57}{14} = \text{say } 500$$

cwt. breaking load in centre, or, allowing a factor of safety of 5, $\frac{500 \times 2}{5} = 200$ cwt. 10 tons = safe load distributed.

Case 2.—A wooden bressummer is required to carry a wall 18 in. thick, weighing, with floors and roof resting on it, 15 tons, over an opening 14 ft. wide.



Figs. 635 and 636.—Halved Joint



Figs. 637 and 638.—Halved Joints.

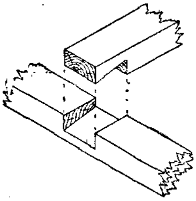


Fig. 639.—Bevelled Halving.

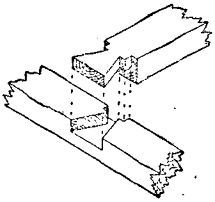


Fig. 640.—Shouldered Dovetail Halving.

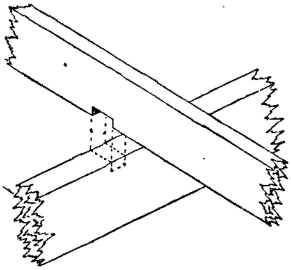


Fig. 641.—Single Notching.

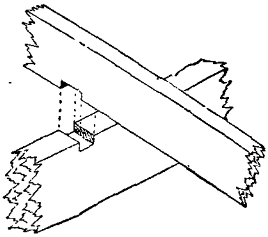


Fig. 642.—Double Notching.

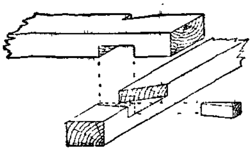


Fig. 643.—Dovetail Notching.

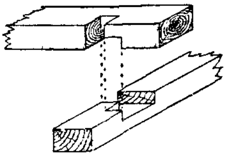


Fig. 644.—Tredgold Notching.

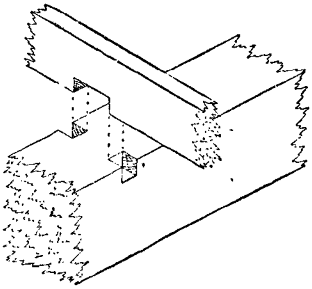


Fig. 645.—Cogging.

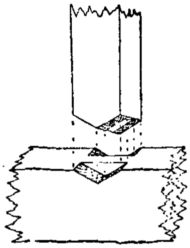


Fig. 647.—Bridle Joint.

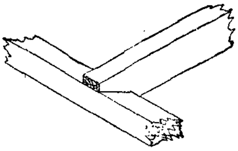


Fig. 646.—Birdsmouthed Joint.

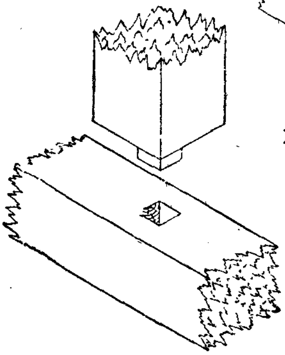


Fig. 650.—Stump or Stub Tenon.

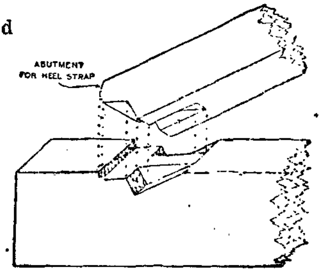


Fig. 648.—Bridle Joint.

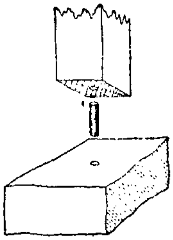


Fig. 649.—Dowelled Post and Sill.

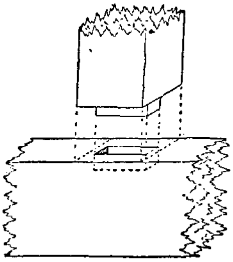


Fig. 651.—Shouldered Tenon.

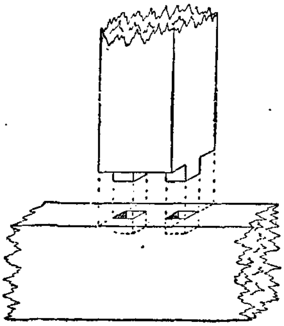


Fig. 652.—Divided Tenon.

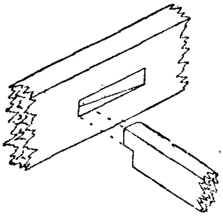


Fig. 653.—Chase Mortice.

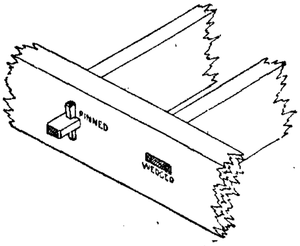


Fig. 655.—Tusk Tenons.

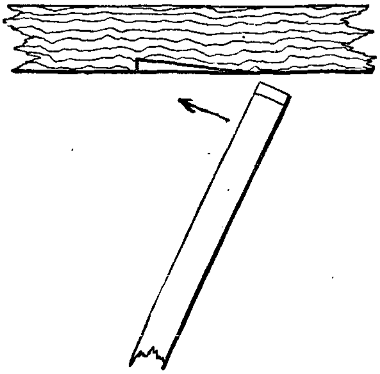


Fig. 654.—Inserting Tenon in Chase Mortice.

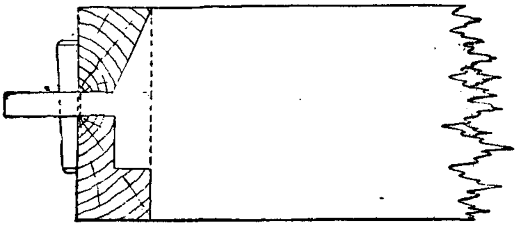


Fig. 657.—Section of Tusk-Tenoned Joint.

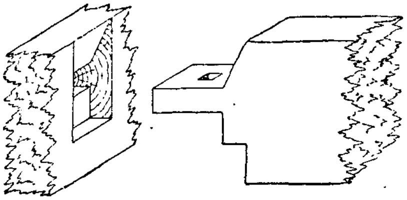
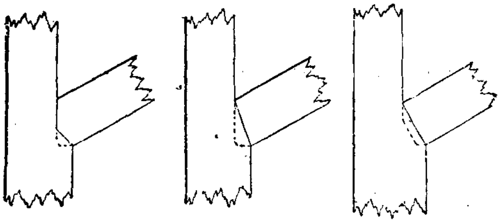
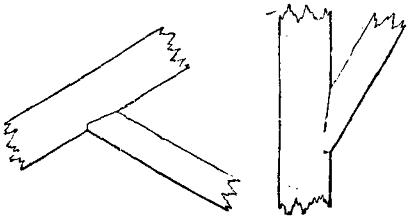


Fig. 656.—Parts of Tusk-Tenoned Joint.



Figs. 658 to 660.—Strut Tenoned into King or Queen Post.



Figs. 661 and 662.—Toe Joint.

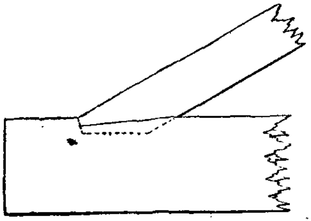
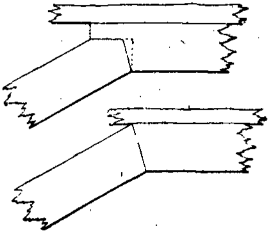


Fig. 663.—Toe Joint with Tenon.



Figs. 664 and 665.—Joints for Gantry Strut.

Formula for simple beam: B.W. cwt. centre $= \frac{c\,b\,d^2}{L}$, whence safe load in tons on fir $= \frac{2 \times 3{\cdot}5 \times b\,d^2}{10 \times 20 \times L}$, or $15 = \frac{2 \times 3{\cdot}5 \times b\,d^2}{10 \times 20 \times L}$ $\therefore b\,d^2 = \frac{15 \times 10 \times 20 \times 14}{2 \times 3{\cdot}5} = 6000$, and $\sqrt[3]{6000} = 18$ in. for side of square beam. This could be obtained in pitch pine only, and it would be very unsuitable for building an 18-in. brick wall upon; a flitched beam must therefore be tried. Say a flitched beam of three 12-in. by 6-in. half timbers and two 12-in. by $\frac{3}{4}$-in. flitched plates.

In designing beams all dimensions are tentative; they are first assumed as near as one's experience can fix them, and their sufficiency or otherwise is then put to the test.

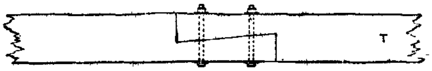


Fig. 666.—Dovetailed Halving.

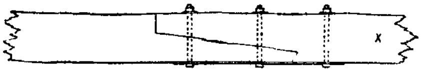


Fig. 667.—Raking Scarf with Butt End.

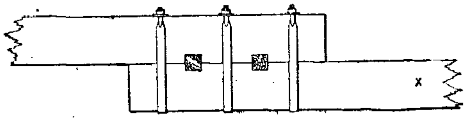


Fig. 668.—Lapped Joint, with Keys and Straps.

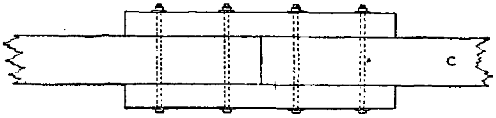


Fig. 669.—Common Fished Joint.

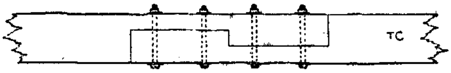


Fig. 670.—Tabled Joint.

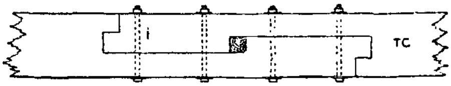


Fig. 671.—Tabled Scarf with Folding Wedges.

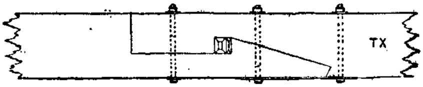


Fig. 672.—Tabled and Splayed Scarf.

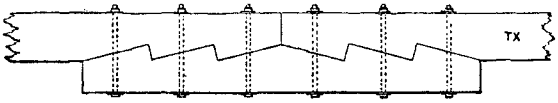


Fig. 673.—Indented Beams for Lengthening and Strengthening.

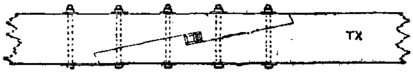


Fig. 674.—Splayed Scarf with Folding Wedges.

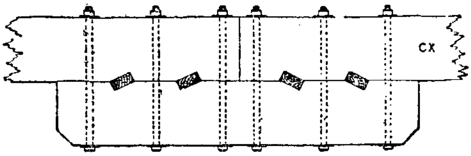


Fig. 675.—Fished Joint, with Oblique Keys.

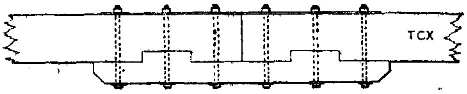


Fig. 676.—Fished and Tabled Joint.

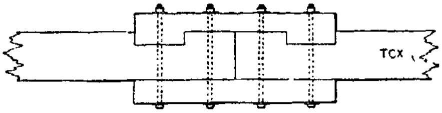


Fig. 677.—Fished and Tabled Joint.

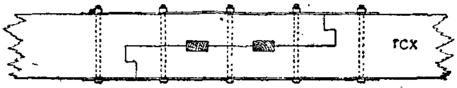


Fig. 678.—Tabled Scarf with Keys and Plates.

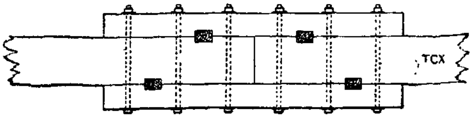


Fig. 679.—Fished Joint, Keyed and Bolted.

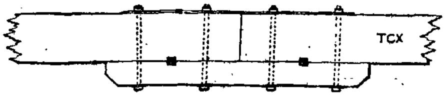


Fig. 630.—Fished Joint with Hardwood Keys.

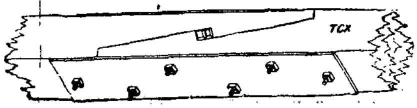


Fig. 681.—Vertical Scarf.

Formula for flitched beams—

$$\text{B.W. cwt. centre} = \frac{d^2}{L}(cb + 30t)$$

$$= \frac{12^2}{14}(3{\cdot}5 \times 3 \times 6 + 2 \times 30 \times \tfrac{3}{4})$$

$$= \frac{144}{14}(63 + 45)$$

$$= \frac{144 \times 108}{14} = 1111 \text{ cwt.}$$

$\frac{1111 \times 2 \text{ for load distrib.}}{20 \text{ cwt. in 1 ton} \times 10 \text{ fact. safety}} = 11{\cdot}11$ tons, which is not nearly enough. Try three flitches of 14 in. by 6 in., and two flitch plates 14 in. by $\frac{3}{4}$ in. Then $\frac{14}{14}(3{\cdot}5 \times 3 \times 6 + 2 \times 30 \times \frac{3}{4})$

$$= 14(63 + 45) = 1512$$

and $\frac{1512 \times 2}{20 \times 10} = 15{\cdot}12$ tons, which will be satisfactory, and the design will be as shown in Fig. 634.

Joints in Carpentry.

The simplest joints used in carpentry are the various forms of halving: simple halved joints (Figs. 635 to 637); dovetail halving (Fig. 638); bevelled halving (Fig. 639); and shouldered dovetail halving (Fig. 640). Of the many forms of notching, there are: single notching (Fig. 641); double notching (Fig. 642); dovetail notching (Fig. 643); and Tredgold notching (Fig. 644). Cogging is shown by Fig. 645, the birdsmouthed joint by Fig. 646, the bridle joint by Figs. 647 and 648, and dowelling of wood to stone by Fig. 649. Of tenon joints, there is the stump, or stub tenon (Fig. 650); the shouldered tenon (Fig. 651); the divided tenon (Fig. 652); the chase mortice (Fig. 653), in the side of a timber, with one cheek cut away and the depth gradually tapering out to the face of the timber. It is used in framed and double floors, for enabling short joists, such as ceiling joists between the binders, to be got into place after the larger timbers are fixed, as in plan Fig. 654. The tusk tenon is shown by Figs. 655 to 657; struts tenoned into king or queen posts are shown by Figs. 658 to 660. Simple toe joints are shown by Figs. 661 and 662, and a toe joint with tenon by Fig. 663. Joints for a gantry strut are shown by Figs. 664 and 665.

Joints for Lengthening Beams and Posts.

A joint suitable for tension only is the dovetailed halving (Fig. 666). A joint suitable for compression only is the common fished joint (Fig. 669). Joints suitable for cross strain only are as follows:—Lapped, with keys and straps (Fig. 668); and the raking scarf with butt end

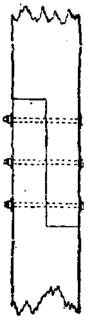


Fig. 682.—Halved Joint.

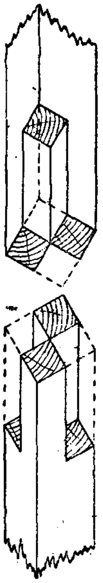


Fig. 683.—Double Halved Joint.

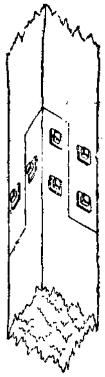


Fig. 684.—Double Halved Joint.

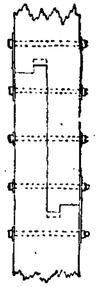


Fig. 685.—Parallel Scarf with Birdsmouthed Ends.

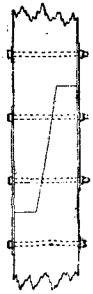


Fig. 686.—Splayed Scarf.

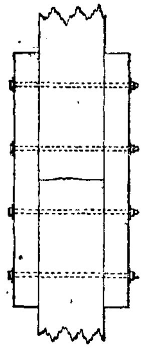


Fig. 687.—Single Fished Butt Joint when Post is Braced.

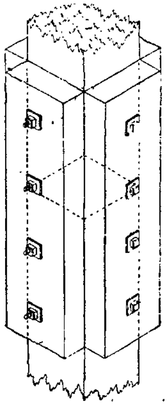


Fig. 688.—Double Fished Butt Joint for Detached Post.

(Fig. 667). Joints suitable for tension and compression are as follows :—Tabled (Fig. 670); and the tabled scarf with folding wedges (Fig. 671). Joints suitable for tension and cross strain are as follows :—Tabled and splayed scarf (Fig. 672); indented beams for lengthening and strengthening (Fig. 673); and the splayed scarf with folding wedges and iron plate covering joint on tension side (Fig. 674). A joint suitable for compression and cross strain is the fished joint with oblique keys (Fig. 675). Joints suitable for tension, compression and cross strain are as follows :—Fished and tabled (Figs. 676 and 677); tabled scarf with keys and plates (Fig. 678); fished, keyed and bolted (Fig. 679); fished, with hardwood keys (Fig. 680); and the vertical scarf (Fig. 681) which is used in the warehouses at the South West India Dock, London. Other joints used for lengthening beams and posts are : The halved joint (Fig. 682); double halved joint (Figs. 683 and 684); parallel scarf with birdsmouthed ends (Fig. 685); splayed scarf (Fig. 686); single fished butt joint when the post is braced (Fig. 687); and double fished butt joint (Fig. 688) when the post is detached.

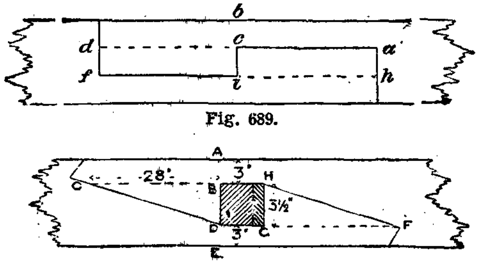
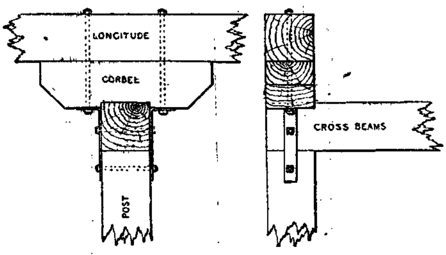


Fig. 689.

Fig. 690.

Figs. 689 and 690.—Diagrams showing Proportions of Scarf Joints.



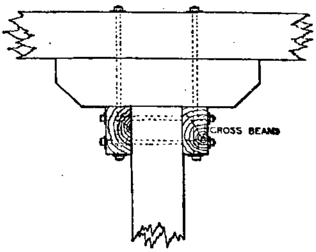


Fig. 695. Fig. 696. Fig. 697.

Figs. 695 to 697.—Beam and Post Joints.

Rule for Proportioning Parts of Scarf.

Tredgold gives the following practical rules for proportioning the different parts of a scarf according to the strength possessed by the kind of timber in which it is formed, to resist tensional, compressional, or shearing forces respectively. In Fig. 689 *c d* must be to *c b* in the ratio that the force to resist detrusion bears to the direct cohesion of the material—that is, in oak, ash, elm, *c d* must be equal to from eight to ten times *c b* ; in fir and other straight-grained woods *c d* must be equal to from sixteen to twenty times *c b*. The sum of the depth of the indents should be equal to one and one-third depth of beam. The length of scarf should bear the following proportion to the depth of the beam :—

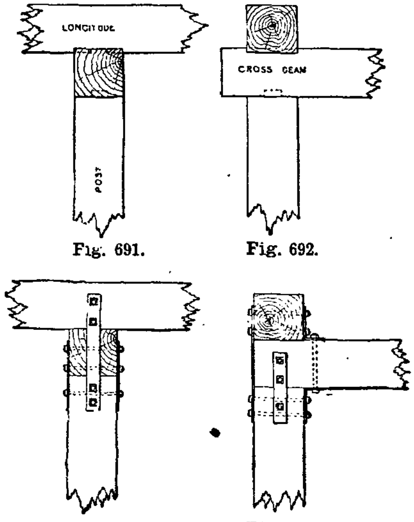


Fig. 691. Fig. 692.

Fig. 693. Fig. 694.

Figs. 691 to 694.—Beam and Post Joints.

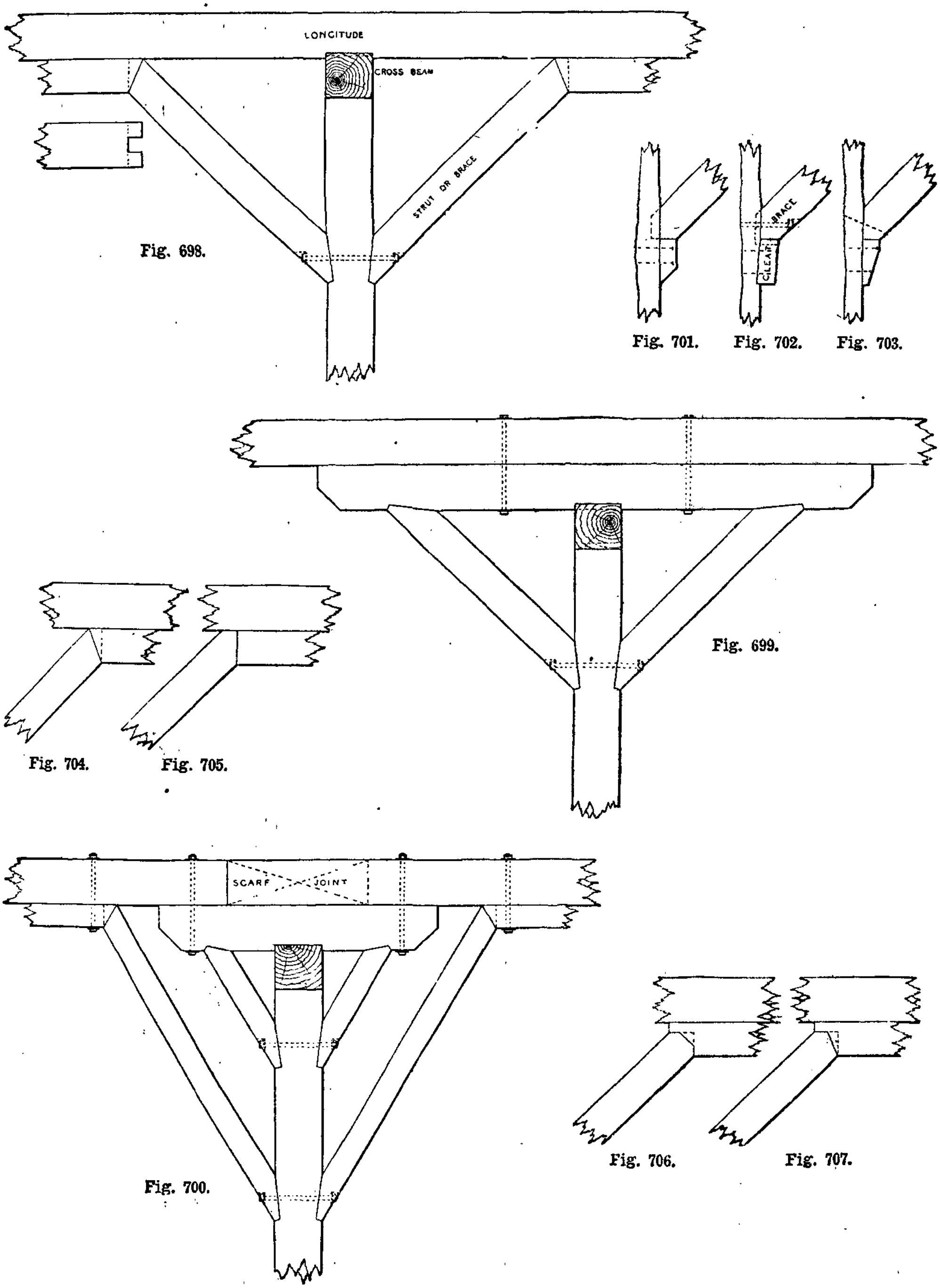

LONGITUDE
CROSS BEAM
STRUT OR BRACE
BRACE
CLEAR
SCARF
JOINT

Fig. 698.

Fig. 701. Fig. 702. Fig. 703.

Fig. 699.

Fig. 704. Fig. 705.

Fig. 700.

Fig. 706. Fig. 707.

Figs. 698 to 707.—Beam and Post Joints.

Wood Used.	Without Bolts.	With Bolts.	With Bolts and Indents.
Hardwood (oak ash, elm) ...	6 times	3 times	2 times
Fir and other straight-grained woods	12 "	6 "	4 "

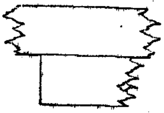


Fig. 708.—Square Corbel End.

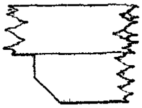
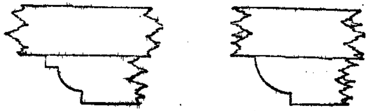


Fig. 709.—Chamfered Corbel End.



Figs. 712 and 713.—Ovolo Moulding on Corbel End.

Calculation of a scarfed joint as Fig. 690:

Per sq. in.
Working resistance to tearing = 12 cwt.
" " compression = 10 "
" " shearing = 1·3 "

Load equals, say, 360 cwt. direct tension beyond that taken by any bolts or plates. The joint may tear across A B or D E, therefore section at A B must equal $\frac{360}{12} = 30$, say, 10 in. by 3 in. The joint may also shear across B C or G F, therefore section at B C or G F must equal $\frac{360}{1 \cdot 3} = 277$, say, 28 in. by 10 in. The joint may also be crushed at B D or G H, therefore section at B D or G H must equal $\frac{360}{10} = 36$, say 10 in. by $3\frac{1}{2}$ in. Thus the beam should be about 10 in. by 10 in., with wedges as shown; but in ordinary practice the folding wedges do not exceed one-fourth the depth of the beam, and are usually placed square to the rake of the scarf, the scarf being further strengthened by bolts and plates.

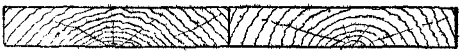


Fig. 716.—Butt Joint.

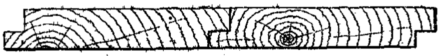


Fig. 717.—Rebated Joint.

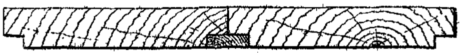


Fig. 718.—Rebated and Filleted Joint.

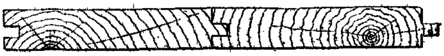


Fig. 719.—Grooved and Tongued Joint.

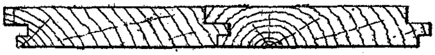


Fig. 720.—Rebated, Grooved and Tongued Joint.

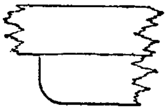


Fig. 710.—Round-nosed Corbel End.

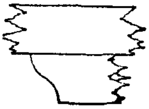


Fig. 711.—Cyma Reversa Moulding on Corbel End.

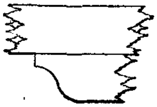


Fig. 714.—Cyma Recta Moulding on Corbel End.

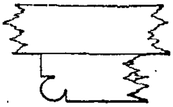


Fig. 715.—Staff or Return Bead on Corbel End.

Strength of Joints in Struts and Beams.

If two deals are bolted together, with distance pieces between, they will be stronger than a solid timber strut of the same sectional area, because the dimension of "least width" in the formula for calculation of strength will be increased. There would be no appreciable advantage in making the distance pieces of different thicknesses, to swell or reduce the middle diameter, they should be all alike, and enough to make the combined thickness not

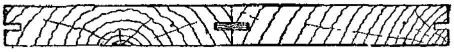


Fig. 721.—Ploughed and Cross-Tongued Joint.

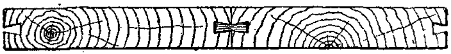


Fig. 722.—Dovetail Slip-Feather Joint.

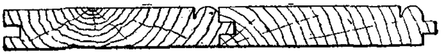


Fig. 723.—Matched and Beaded Joint.

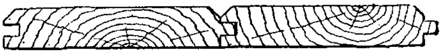


Fig. 724.—Matched and Vee-jointed.

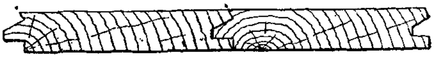


Fig. 725.—Splay-Rebated Joint.

less than three-fourths of the width of the deals, and the distance apart in feet should be equal to the length of the deal in feet multiplied by its thickness in inches and divided by the width in inches. Single ½-in. bolts are of no use in rough carpentry, except for very small work ; instead, two ⅝-in. bolts should be placed diagonally through each block. Horizontal

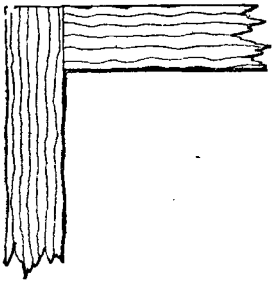


Fig. 726.—Plain Butt Joint.

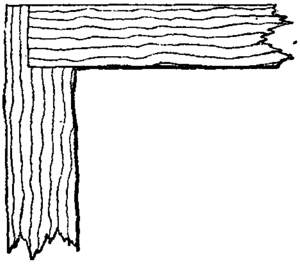


Fig. 727.—Rebated Butt Joint.

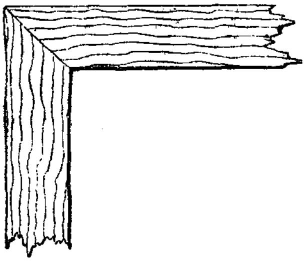


Fig. 728.—Plain Mitre Joint.

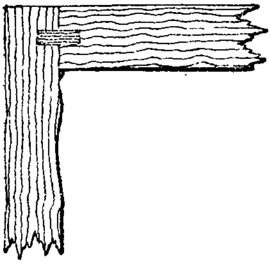


Fig. 729.—Butt Joint Tongued.

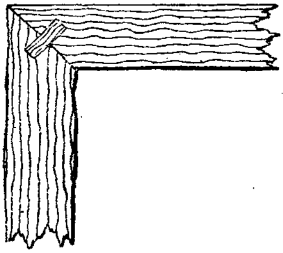


Fig. 730.—Mitre Joint Tongued.

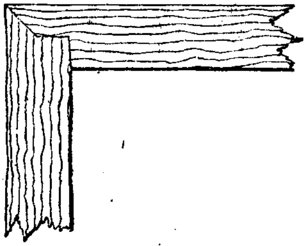


Fig. 731.—Rebated and Mitred Joint.

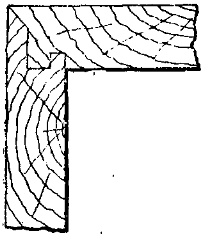


Fig. 732.—Mitred, Grooved, and Tongued Joint.

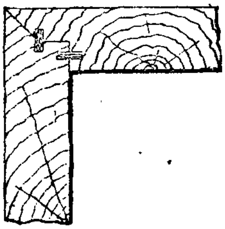


Fig. 733.—Rebated, Mitred, and Double-Tongued Joint.

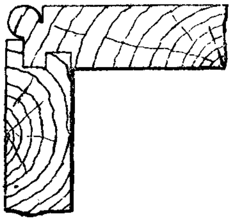


Fig. 734.—Rebated, Tongued, and Staff Beaded Joint.

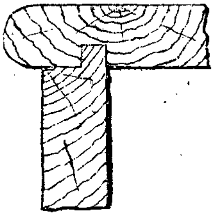


Fig. 735.—Rebated and Grooved Joint to Nosing.

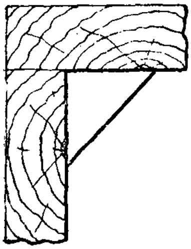


Fig. 736.—Glued Blockings.

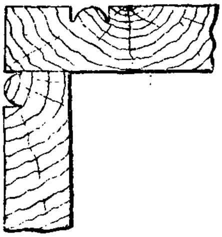


Fig. 737.—Butt Joint with Flush Beads.

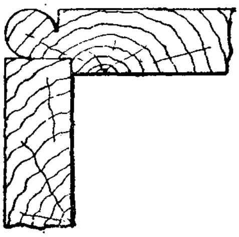


Fig. 738.—Rebated and Staff Beaded Joint.

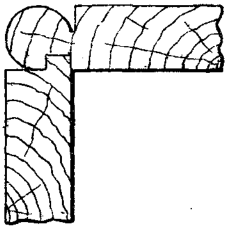


Fig. 739.—Rebated, Grooved, and Staff Beaded Joint.

connecting rods in machinery are sometimes swelled in the middle to allow for the cross strain upon them in addition to the end-long strain, while vertical struts have no cross strain to meet. In Classic architecture the columns are swelled in the middle for appearance only.

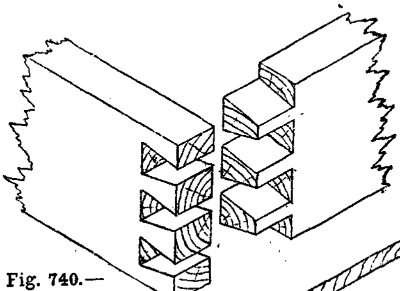


Fig. 740.—Dovetail Joint.

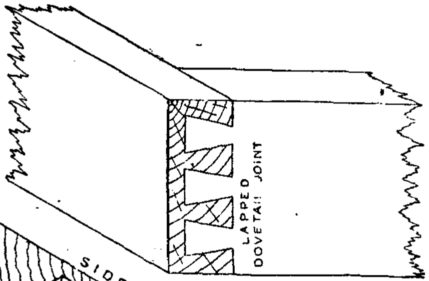


Fig. 741.—Lapped Dovetail Joint.

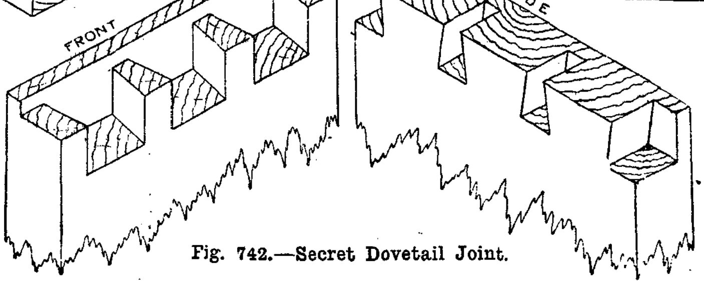


Fig. 742.—Secret Dovetail Joint.

Jointing Beams to Posts.

The usual methods of forming joints between posts and beams are illustrated by Figs. 691 to 707. The wording on the illustrations makes the methods quite clear to understand. Ends of corbels are illustrated by Figs. 708 to 715.

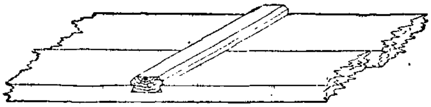


Fig. 743.—Dovetail Ledged.

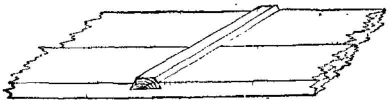


Fig. 744.—Diminished Dovetail Ledged.

Joints in Joinery.

Ten joints used for connecting boards edge to edge are shown by Figs. 716 to 725. Matchboarding is thin stuff with a tongue and bead worked on one edge and a groove on the other, so that when the pieces are put together the joint is masked by the bead, and the tongue prevents dust and draught from passing through, as in Fig. 723. A slip feather is a piece of wood inserted in plough grooves, as in Fig. 721, to strengthen a glued joint, or to keep out dust. It may be of soft wood, and is then in short lengths, made by cutting pieces 1 in. wide off the end of a plank, turning the pieces over and cutting them into thin strips, with the grain across their length. If hard wood is used, the grain may run in the direction of the length. The slip feathers may also be double, or dovetailed. Fourteen styles of angle joints are shown by Figs. 726 to 739. Dovetail joints are known in great variety, but it will be sufficient to show the ordinary dovetailing (Fig. 740), lapped dovetail (Fig. 741), and the secret dovetail (Fig. 742). The dovetail ledged and the diminished dovetail ledged are shown respectively by Figs. 743 and 744. The ordinary dowelled joint is represented by Fig. 745.

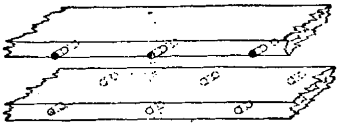


Fig. 745.—Dowelled Joint.

The simple housing joint is shown by Fig. 746. Some tenon joints have already been shown under the heading "Joints in Carpentry" (p. 177); further tenon joints more especially used in joinery are: The simple tenon and mortice (Fig. 747); pair of single tenons, commonly called "double tenons" (Fig. 748); double or twin tenons (Fig. 749); pair of single tenons with grooves and slip feathers (Fig. 750); haunched tenon (Fig. 751); dovetail tenon (Fig. 752); pinned tenon (Fig. 753); tusk tenons and stump or stub tenons are also used in joinery, and have already been illustrated (Figs. 650 and 656). The foxtail tenon (Fig. 754) is a good joint; alternative methods (with and without housing) of applying this in fitting rails into an oak gate post are shown by Fig. 755. There is no universal rule for proportioning tenons, but the practice is to give from half to the whole of the width of the rail for the width of the tenons. If more space than half were given to a haunched tenon, the end of the stile would be liable to be driven out in wedging up; and there is no reason why more space should be given. With regard to the application of the various tenon joints, a few of these are noted below: A simple tenon, one-third thickness of the stuff, is used in framing together pieces of the same size, the mortice being just long enough to allow of a wedge being driven in on each side of the tenon to secure it. A pair of single tenons,

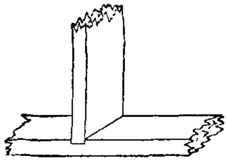


Fig. 746.—Housing Joint.

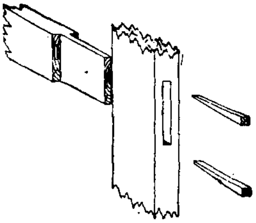


Fig. 747.—Tenon and Mortice Joint.

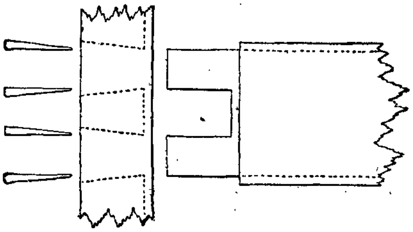


Fig. 748.—Pair of Single Tenons.

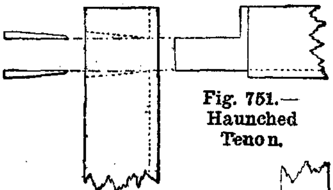


Fig. 751.—Haunched Tenon.

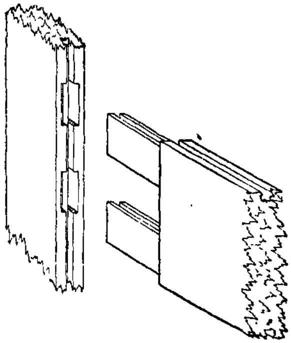


Fig. 749.—Double or Twin Tenons.

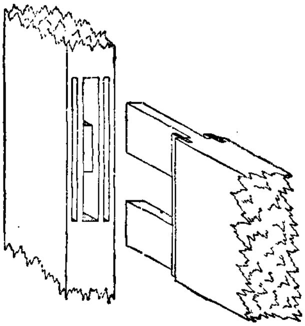


Fig. 750.—Pair of Single Tenons with Grooves and Slip Feathers.

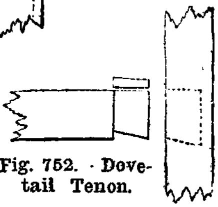


Fig. 752. · Dovetail Tenon.

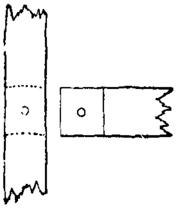


Fig. 753.—Pinned Tenon.

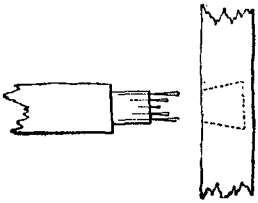


Fig. 754.—Foxtail Tenon.

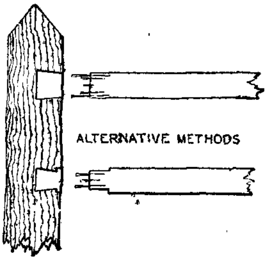


Fig. 755.—Foxtail Tenons with and without Housing.

usually called a double tenon, is used for connecting the middle rail of a door to the stiles. A haunched tenon for connecting the top rail of a door to the stiles; the tenon being half the width of the top rail leaves a haunch or haunching to prevent the rail from twisting. A stump or stub tenon is used at the foot of a post to prevent movement. A tusk tenon is used in framing trimmers to trimming joists, to obtain the maximum support with the minimum reduction of strength. A tenon with only one shoulder is used in framed and braced batten doors, and in skylights, when the rail requires to be kept thin for other parts to pass over. A pair of double tenons is used for the lock rail of a thick door, to receive a mortice lock. The hammer-headed key joint is shown in plan and section by Figs. 756 and 757.

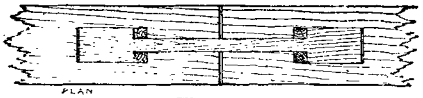


Fig. 756.

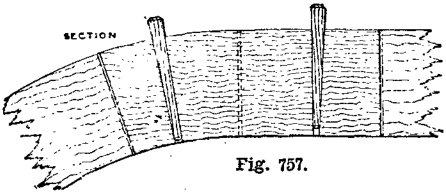


Fig. 757.

Figs. 756 and 757.—Hammer-headed Key Joint.

Construction of Floors.

Single Floors.—A single floor is shown in section by Fig. 758 on this page. This is a ground floor, the section being taken at right angles to the herringbone strutting. A single floor of 18 ft. span may be of 11 in. by 3 in. joists stiffened with three rows of herringbone strutting.

Double Floors.—Two sections through an upper floor—a double floor—are presented by Figs. 759 and 760. Just above the lath-and-plaster ceiling are what are known as the ceiling joists. Running parallel with these is a 10-in. by 5-in. rolled iron joist (the binder). The bridging, or common wooden joists shown in elevation in Fig. 762, run at a right angle to the iron binder and ceiling joist. A section through another double floor is presented by Fig. 761; the girder measures 12 in. by 6 in., the bridging joists 8 in. by 3 in., and the floor boards are 1½ in. thick and have splayed headings; if constructed with binders (described later) this floor would become a framed floor. The floor shown by Fig. 762 has binders, but is not a framed floor; it is known as a double floor with rebated and filleted battens, and it

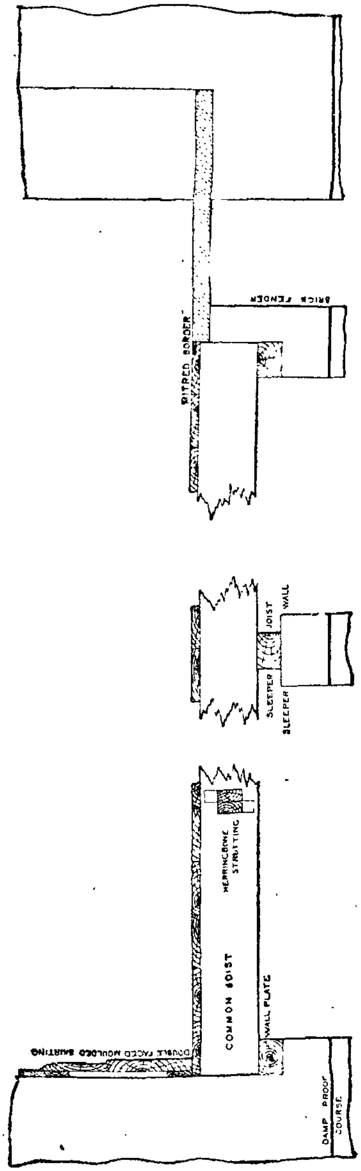


Fig. 758.—Section through Ground (Single) Floor.

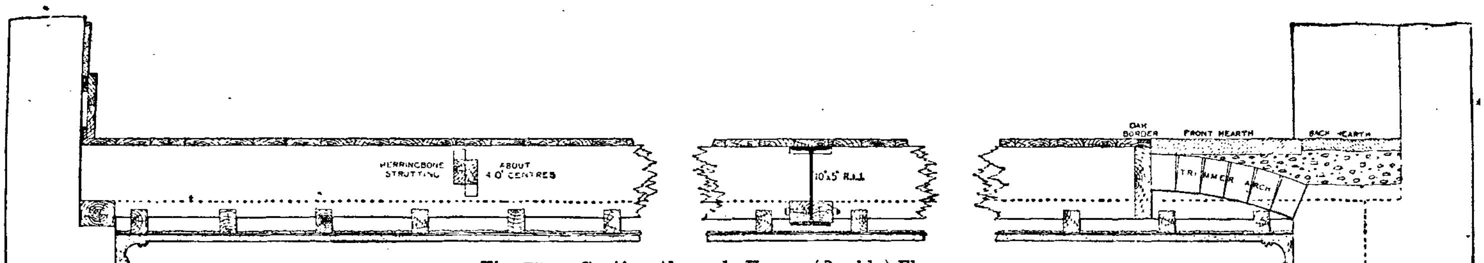

HERRINGBONE STRUTTING
ABOUT 4 0' CENTRES
10"x5" R.J.
OAK BORDER
FRONT HEARTH
BACK HEARTH
TRIMMER ARCH

Fig. 759.—Section through Upper (Double) Floor.

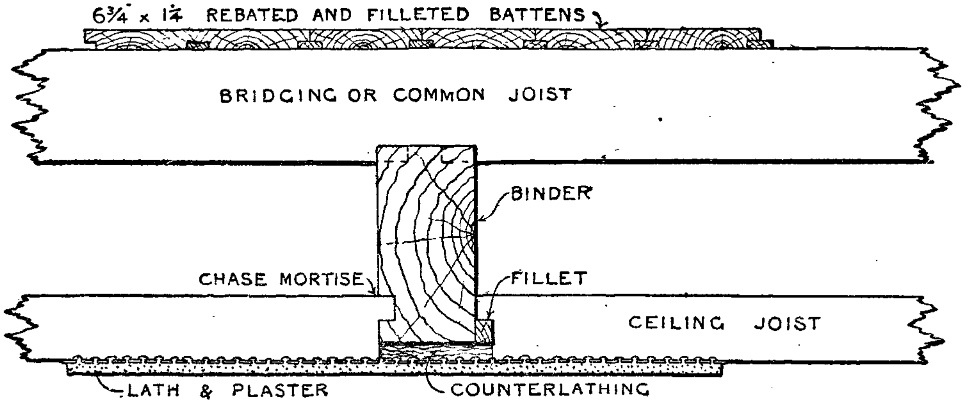

6¾" x 1¼" REBATED AND FILLETED BATTENS
BRIDGING OR COMMON JOIST
BINDER
CHASE MORTISE
FILLET
CEILING JOIST
LATH & PLASTER
COUNTERLATHING

Fig. 762.—Section through Double Floor with Rebated and Filleted Battens. Scale $\frac{3}{32}$ full size.

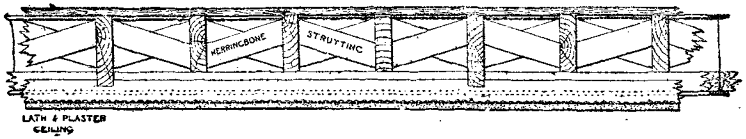

HERRINGBONE
STRUTTING
LATH & PLASTER CEILING

Fig. 760.—Section across Joists showing Herringbone Strutting.

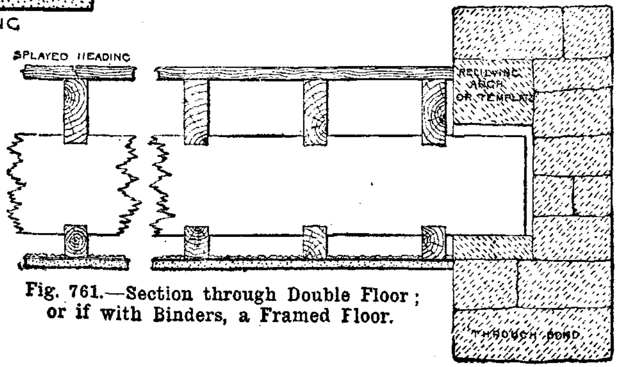

SPLAYED HEADING
RELIEVING ARCH OR TEMPLATE
THROUGH BOND

Fig. 761.—Section through Double Floor; or if with Binders, a Framed Floor.

should be compared with later illustrations (Figs. 764 and 765 on this page).

Framed Floors.—A framed floor is built up as shown in the isometric view (Fig. 763), in which the names of the joists, etc., are clearly given. Cross sections taken at a right angle to one another are given in Figs. 764 and 765; these represent a framed floor with ploughed and cross-tongued boards.

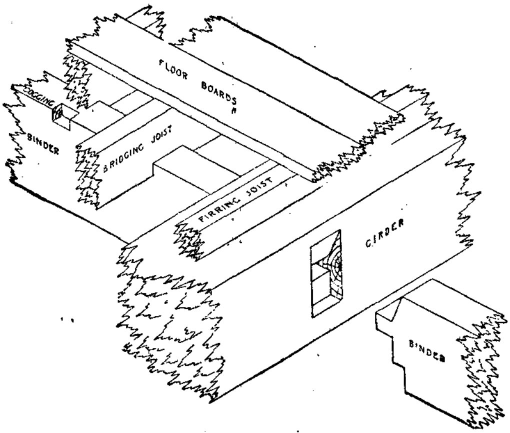

FLOOR BOARDS
COGGING
BINDER
BRIDGING JOIST
FIRRING JOIST
GIRDER
BINDER

Fig. 763.—Part of Framed Floor.

Wooden Binders.—The sections of wood binders given in Figs. 762 and 765 show the general methods of connecting them to the bridging and ceiling joists. Fig. 766 shows a 6-in. by 3-in. bridging joist cogged to a binder, and

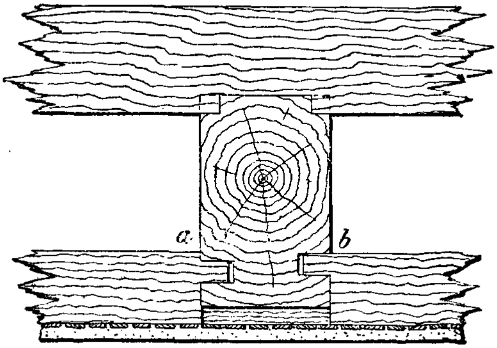


Fig. 766.—Bridging Joist, Ceiling Joist, and Binder.

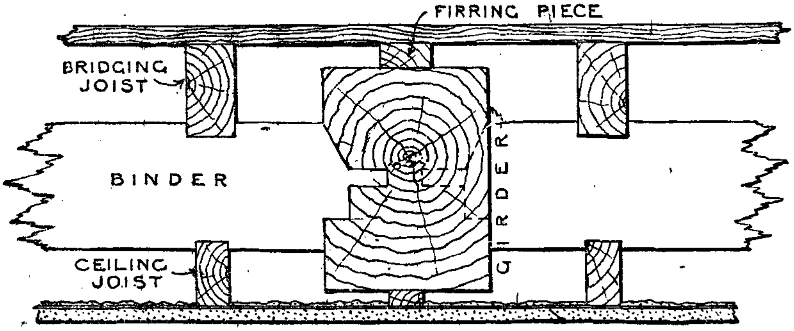

FIRRING PIECE
BRIDGING JOIST
BINDER
GIRDER
CEILING JOIST

Fig. 764.—Section through Girder of Framed Floor.

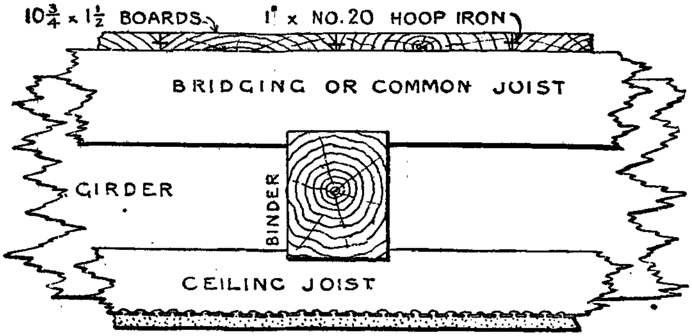

10¾ × 1½ BOARDS
1" × NO.20 HOOP IRON
BRIDGING OR COMMON JOIST
GIRDER
BINDER
CEILING JOIST

Fig. 765.—Section through Binder of Framed Floor with Ploughed and Tongued Boards.

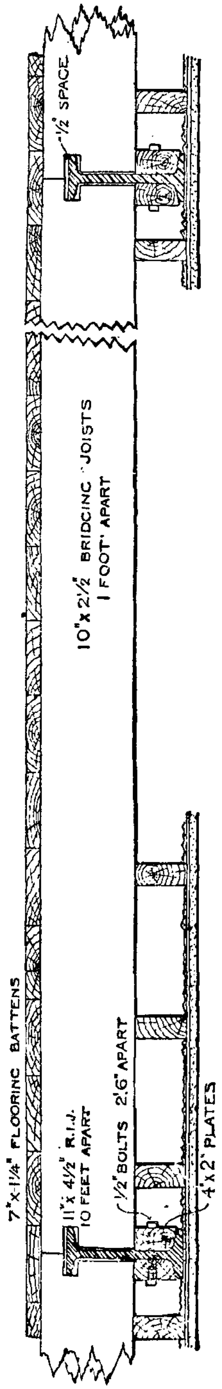

7"×1¼" FLOORING BATTENS
11"×4½" R.I.J.
10 FEET APART
½" BOLTS 2'6" APART
4'×2' PLATES
10"×2½" BRIDGING JOISTS
1 FOOT APART
½" SPACE

Fig. 767.—Section through Rolled-iron Binders of Double Floor.

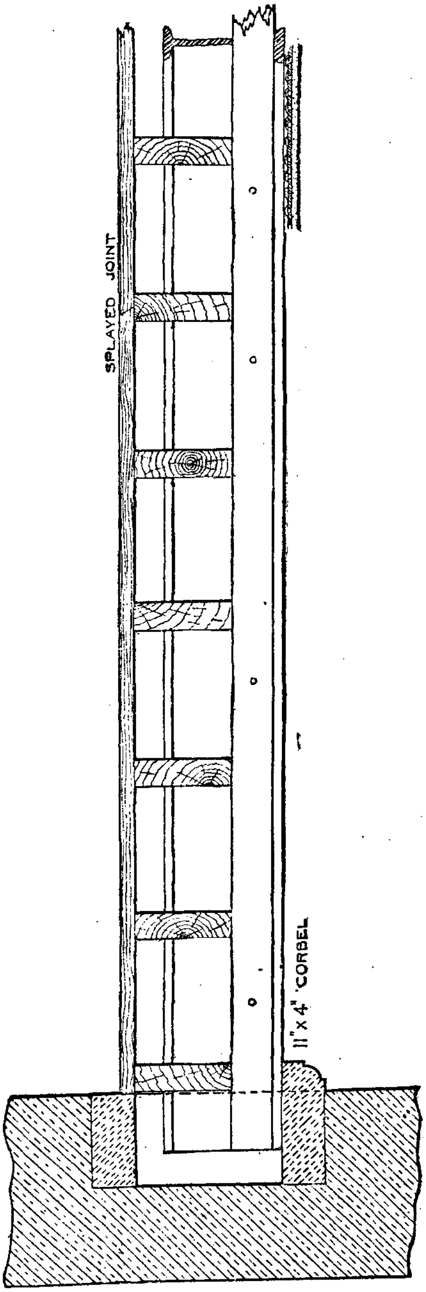

SPLAYED JOINT
11"×4" CORBEL

Fig. 768.—Section through Bridging Joists of Double Floor.

a 4-in. by 2-in. ceiling joist tenoned to it, and carrying a lath-and-plaster ceiling. Alternative arrangements of the tenons are indicated.

Iron Binders.—An iron binder has already been shown (see Fig. 759). In Figs. 767 and 768, now presented, the binders of a double floor are of rolled iron 11 in. deep and 4½ in. wide in the flanges, and 10 ft. apart. Fig. 767 is a cross section of a portion of the floor covering a little more than two binders, and Fig. 768 is a section at right angles to the first showing wall end of binder, which rests on a projecting stone corbel. The bridging joists are of wood.

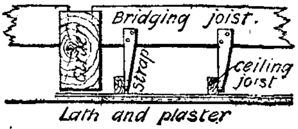

Bridging joist.
Strap
ceiling joist
Lath and plaster

Fig. 769.—Ceiling Joists.

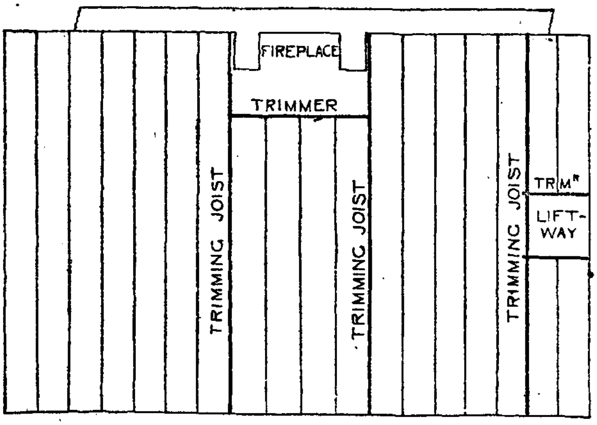

FIREPLACE
TRIMMER
TRIMMING JOIST
TRIMMING JOIST
TRIMMING JOIST
TRIMᴹ
LIFT-WAY

Fig. 770.—Joists, etc., round Fireplace and Lift-way.

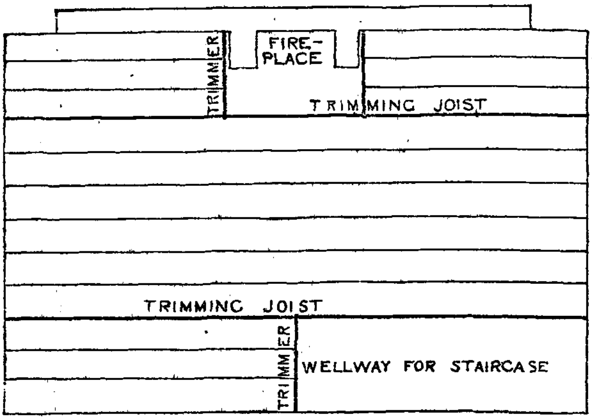

TRIMMER
FIRE-PLACE
TRIMMING JOIST
TRIMMING JOIST
TRIMMER
WELLWAY FOR STAIRCASE

Fig. 771.—Joists, etc., round Fireplace and Staircase Well.

Hanging Ceiling Joists.—Ceiling joists are sometimes hung by straps of wood from the floor joists, in the manner shown in Fig. 769, instead of being fixed direct to the under side of the joists. A strap is required at every passing of every joist. Ceiling joists are mostly used with double and framed floors, as shown in Figs. 760 to 767; but may be used with single floors, if desired, the object being to obstruct the passage of sound and to stiffen the floor. The ordinary method of rendering a floor sound-proof is shown on the next page.

Scantlings for Joists.

Common joists are spaced 12 in apart, with herringbone strutting every 4 ft. Dimensions for common joists are as follow:—

Span or Length of Bearing in Feet.	Depth in Inches.			
	1¾ in. thick.	2 in. thick.	2½ in. thick.	3 in. thick.
6	6	5¾	5⅜	5
8	7½	7	6½	6¼
10	8½	8	7½	7
12	9¾	9½	8½	8
14	10½	10	9½	9
16	11½	11	10½	10

The nearest available size should be used, and 2-in. ceiling joists should be ½ in. deep per foot span. The trimming joist is made ⅛ in. thicker for every common joist carried by the trimmer.

Formula for Determining Size of Joists.

The following is the formula for determining the size for the joists of a common floor 15-ft. span:—Formula for strength of beam (fir): Breaking weight, cwt., centre $= 4\frac{bd^2}{L} \times 2$ for breaking weight distributed, divided by 10 for safe load. Therefore the safe load cwts. distributed $= \frac{4 \times 2}{10} \times \frac{bd^2}{L} = {\cdot}8\frac{bd^2}{L}$. Assume joists 12-in. centres, then 15-ft. span by 1¼ cwt. per foot run $= 18{\cdot}75$ cwt., whence $18{\cdot}75 = {\cdot}8\frac{bd^2}{L}$. Assume $b = 3$ in., then $d = \sqrt{\left(\frac{18{\cdot}75 \times 15}{{\cdot}8 \times 3}\right)}$ $= \sqrt{117{\cdot}2} = 10{\cdot}82$, say 11 in. × 3-in. joists. Hurst's "Pocket Book" gives a table of scantlings for single joists of Baltic pine, in which 9 × 3 is shown for 14-ft. span, and 11 × 2 for 16-ft. span, both being placed at 12-in. centres. Tredgold's formula for fir joists, 12-in. centres, is $d = \sqrt[3]{\left(\frac{L^2}{b}\right)} \times 2{\cdot}2 = 9{\cdot}28$, say 9¼ × 3 for 15-ft. span. Probably 9 in. by 3 in. would be sufficient for bedrooms, and 11 in. by 3 in. would be desirable for reception-rooms.

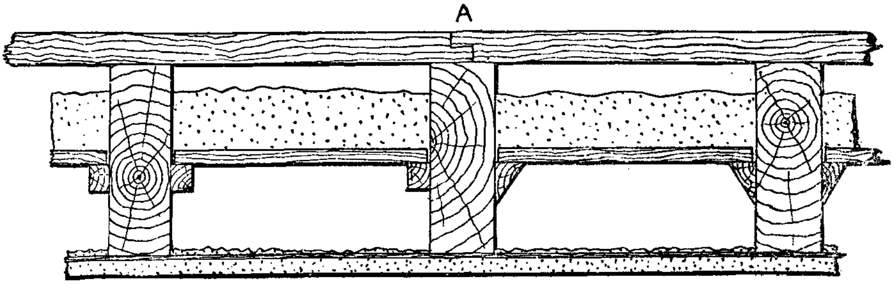
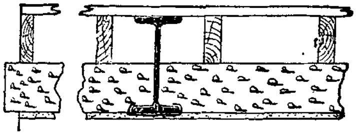


Fig. 772.—Section of Sound-proof Floor with Pugging.



Figs. 773 and 774.—Sound-proof Floor with Concrete

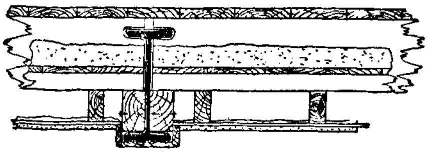


Fig. 775.—Sound-proof Floor with Pugging and Steel Joists.

It will be noticed that the calculations in books are usually based upon the assumption that the joists are placed 12 in. centre to centre, while the ordinary practice is to place them 12 in. apart.

Joists and Trimming Round Openings.

Sketch plans showing the arrangements of joists and trimming round openings are presented by Figs. 770 and 771. In Fig. 770 allowance has to be made for a fireplace and lift-way, and in Fig. 771 for a fireplace and a well-way for a staircase.

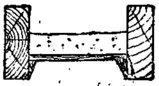
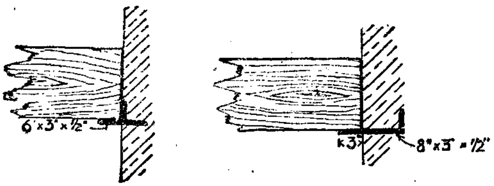


Fig. 776.—Section through Pugging.

Sound-proof Floors.

Fig. 772 shows the section of part of a common floor, showing 9-in. by 3-in. joists, and 1¼ in boarding with a rebated heading joint. In addition, "pugging" and a lath-and-plaster ceiling are shown. The object of the pugging is



Figs. 777 and 778.—Ends of Joist resting on Tee-Iron and Angle-Iron respectively.

to reduce the transmission of sound. Alternative sections of the fillets for supporting the pugging are given in Fig. 772. Take the following case: A hall in a girls' school is 64 ft. by 24 ft.; it is divided into three equal class-rooms by movable partitions; the floor above is partitioned into music-rooms, and it is important that sound shall not pass through the floor and ceiling, which are carried on rolled joists, lying across the 24-ft. span, and these are managed so as to show below the ceiling as little as possible; no timber enters any wall. The span is 24 ft., therefore for a concrete sound-proof floor, the rolled joists should not be less than 12 in. deep. Rolled steel joists 12 in. by 6 in. by 54 lb. will carry a distributed load of 13·7 tons. Allowing ½ cwt. per square foot for weight of concrete in floor, ¼ cwt. per square foot for weight of remainder of floor, and 1¼ cwt. for external load, making a total of 2 cwt. per foot superficial, each joist could support $\frac{13{\cdot}7 \times 20}{2 \times 24} = 5{\cdot}7$ ft wide; but as the space below is to be divided into three equal portions, the floor may be divided into nine bays of $\frac{64}{9} = 7{\cdot}1$ ft., with a smaller factor of safety. Concrete 6 in. thick will support an average load over this span, so that the floor might be constructed as Figs. 773 and 774. If, however, only a pugged floor is intended, the rolled joists can be spaced to $\frac{13{\cdot}7 \times 20}{1\frac{1}{2} \times 24} = 7{\cdot}6$ ft.; but this will not suit the spacing of the rooms below, which would require say $\frac{64}{6} = 10{\cdot}7$ ft. centre to centre. To do this, 14-in. by 6-in. by 57-lb. rolled steel joists will be required, and the section might be as Figs. 775 and 776, the ends of the joists next the wall resting on a tee-iron built in as shown in Fig. 777, or unequal-sided angle-iron, as shown at Fig. 778.

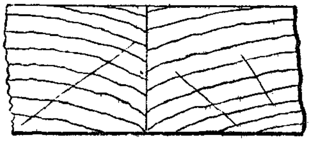


Fig. 779.—Straight Floor Joint.

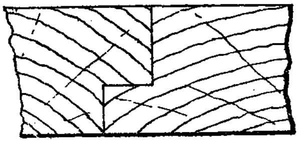


Fig. 780.—Rebated Floor Joint.

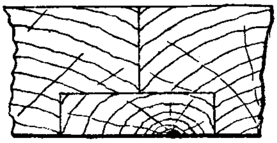


Fig. 781.—Rebated and Filleted Floor Joint.

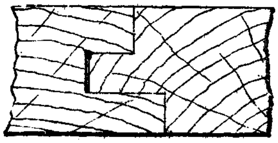


Fig. 782.—Rebated, Grooved and Tongued Joint for Secret Nailing.

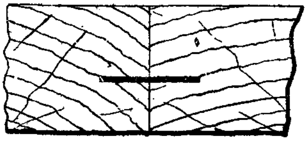


Fig. 783.—Tongued Floor Joint.

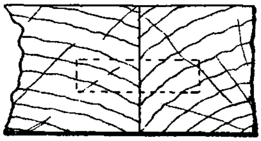


Fig. 784.—Dowelled Floor Joint.

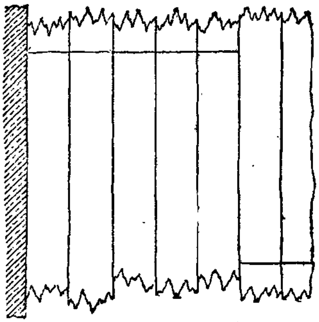


Fig. 785.—Folded Floor.

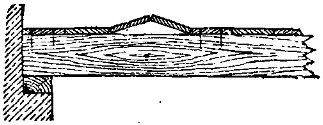


Fig. 786.

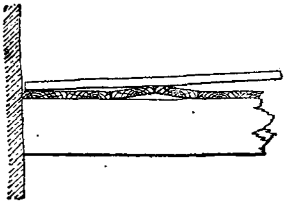
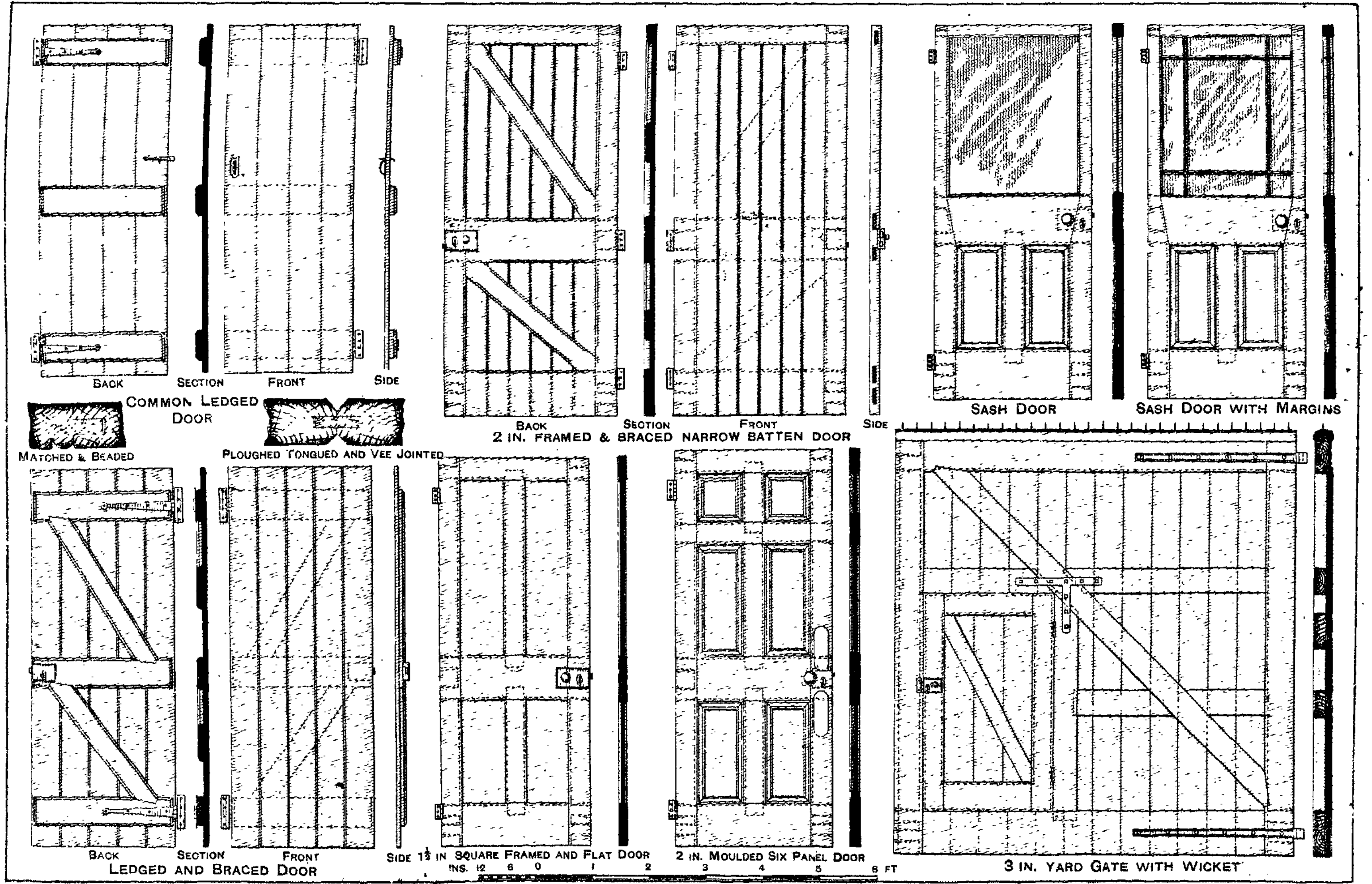


Fig. 787.

Figs. 786 and 787.—Laying Folded Floor.


Back
Section
Front
Side
Common Ledged Door
Matched & Beaded
Ploughed Tongued and Vee Jointed
Back
Section
Front
Side
2 in. Framed & Braced Narrow Batten Door
Sash Door
Sash Door with Margins
Back
Section
Front
Side
Ledged and Braced Door
1½ in Square Framed and Flat Door
2 in. Moulded Six Panel Door
3 in. Yard Gate with Wicket
Ins. 12 6 0 1 2 3 4 5 6 Ft

DOORS.

Joints for Floor Boards.

The ordinary straight joint is shown in section by Fig. 779; the rebated joint (Fig. 780) is another common method, a joint requiring more work being the rebated and filleted (Fig. 781). The rebated, grooved and tongued joint (Fig. 782) is useful for secret nailing. The joint shown in Fig. 783 has an iron tongue, and Fig. 784 shows the dowelled joint. The ploughed and cross-tongued joint with slip feather (Fig. 721, p. 182) is also used.

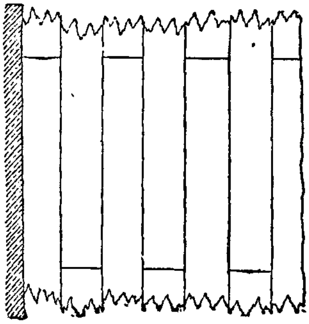


Fig. 788.—Floor with Joints Broken at 3-ft. Intervals.

Laying Floor Boards.

Folded Floor.—The phrase "Breaking joint at every 3-ft. breadth" is common; this applies to boards "laid folding"; that is, one board is laid down to begin with and nailed, then other boards (say five) to make a width of about 3 ft. are laid down (but not nailed) with the ends all in line (see Fig. 785), and the position the outer one (fifth) reaches to is marked on the rafters or joists. The board (fourth) next to the outer one is then taken up, and the outer one is nailed down ¼ in. inside the marks. The fourth board is then put back, but resting on one edge and against the upturned edge of the next (third), as indicated in Fig. 786. These are then forced into place by standing on them or by means of a board as in Fig. 787, which makes the joints close without the use of a cramp, and all the intermediate boards are then nailed.

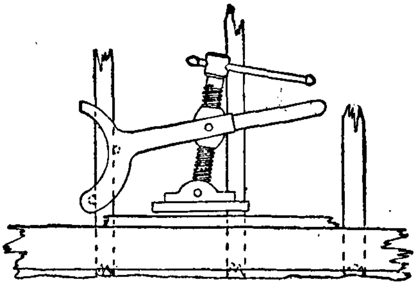


Fig. 789.—Cramping Floor Boards.

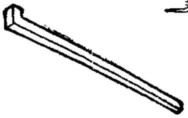


Fig. 790.—Floor Brad.

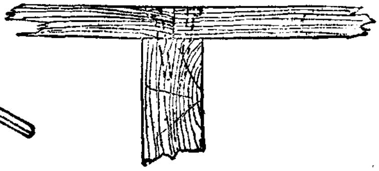
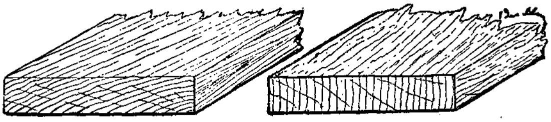


Fig. 791.—Butt Heading Joint in Floor.

Laying Floor Boards with aid of Cramp.—When the joints are broken at 3-ft. intervals as in Fig. 788, a special floor-board cramp has to be used. For instance, batten-width tongued and grooved common Baltic flooring would be laid in the following manner. The joists would be tried over and brought to a level. A batten, or line of battens, would be laid down next the wall to line true at the outer edge, and then be nailed to the joists. The remaining rows are laid two or three at the time with the tongues inserted, then cramped into place, nailed, and the next lot of battens applied. If the battens are already tongued, they can be laid either way, as the block, or saving piece, between the cramp and batten can be grooved to clear the tongue. Fig. 789 shows the mode of using the cramp. When the floor has been finished so far that there is not sufficient room for the cramp, the remaining battens can be wedged in from the wall, or forced together by using a piece of quartering as a lever.

Floor Brads.—Nails used in flooring are called floor brads (Fig. 790), and they are driven through the floor boards into the joists, two at each passing, about 1 in. from the edge. Fig. 791 shows cross section of a butt heading joint, and indicates the manner of inserting the nails.


Figs. 792 and 793.—Directions of Grain in Floor Boards.

Brief Specification.—For best work, under the general clauses will be : "All floor boards must be stacked under cover for seasoning as soon as the buildings are commenced." Under the special clauses will be : "The floor to be laid with 1¼-in. rebated and filleted narrow battens of red deal, with the heading joints rebated, well cramped up, bradded with 2¼-in. floor brads punched in; the surface of the floor to be well cleaned off and nail holes puttied." In common floors the rebating and filleting would be omitted. The lengths should run through if convenient, but nothing less than 8 ft. should be inserted, and, if possible, nothing less than 12 ft., the average being, of course, more than this.

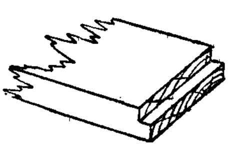


Fig. 794.—Rebate on End of Board.

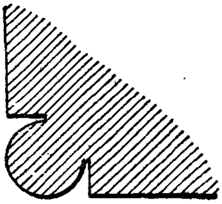


Fig. 795.—Staff Bead.

Direction of Grain in Floor Boards.—If a specification does not insist on any particular position of the grain of the wood, it will be complied with by either of the examples shown in Figs. 792 and 793. If the grain is intended to show "annual rings parallel with the edges," words to that effect should be inserted in the specification, or it should be stated that "all boards are to be cut radially from the tree." There is no doubt that the plank shown in Fig. 793 would be preferable, as being less liable to warp than that shown in Fig. 792 ; but to obtain them would mean picking over a very large parcel of boards in order to get the quantity required, and it may be looked upon as impracticable.

Estimating Load on Floors.

Floors should be estimated for according to the nature of the building and the probable load. A crowd of persons is variously estimated to weigh from 41 lb. to 147·4 lb. per square foot of the surface covered. Probably a safe average would be 1 cwt. per ft. super. considered as a live load. Dwelling houses are usually designed for a dead load of 1¼ cwt. per foot super., churches and public buildings 1½ cwt., and warehouses 2½ cwt. The weight of the structure must be allowed for in addition to the above loads, and this is most important to bear in mind in connection with fireproof floors. For dwelling houses the 1¼ cwt. is usually made to include the weight of the floor itself.

Floor for Dancing Room.

Various methods have been proposed for imparting a certain amount of spring to floors used for dancing, but no general consensus of opinion as to the best means of attaining the desired object is yet available. If the floor is supported on girders or joists they must be strong enough to carry the load, and this necessarily renders the floor somewhat stiff. The stiffness of the floor may, however, be lessened without reducing the ultimate strength by making the joists, etc., shallower, and giving them greater thickness ; but the cost will be increased, as a larger sectional area will be necessary. If concrete is laid on the surface of the ground, and parquet flooring bedded on waterproof mastic laid on the concrete, a floor is produced that is dead, cold, and unresponsive. This feeling of deadness might perhaps be lessened and springiness imparted to the floor if ⅜ in. or ½ in. of impregnated hair felt is laid under the parquet blocks.

Damp Joists on Ground Floor.

If the ground-floor joists and flooring are covered on the under side with drops of water, two conclusions that may be certainly drawn are that there is not enough ventilation, and that the subsoil must be very wet. It should be ascertained whether the site is covered with concrete, whether there is a damp-proof course in the walls, whether the lower part of the walls is damp, whether the moisture is always

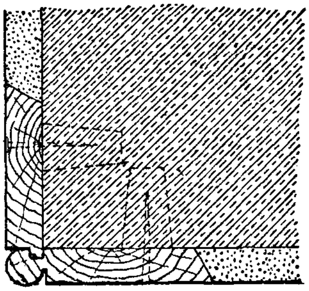


Fig. 796.—Angle Bead fixed to Wall.

present or otherwise, the nature of the soil, and the aspect of the building and its position as regards hill or valley. In an obscure case, no point should be left without investigation; but generally a through current of air will prevent an accumulation of moisture.

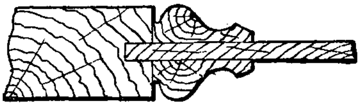


Fig. 797.—Planted Moulding.

Dry Rot in Floor.

When a floor collapses as the result of dry rot or fungoid growth, there is generally no doubt that the evil arises from want of ventilation. Air-bricks should be inserted in the walls, back and front, below the floor, so as to give a through current. All defective timber should be removed at once, and the remainder of the timber, wherever the slightest symptom of fungus appears, as well as the walls, should be well scraped and painted over with a solution of blue vitriol (copper sulphate). The worst part of the floor or wall should be watched for a considerable time, and be scraped and painted again if necessary. Dry rot when it has once got a firm hold of a place is not easily eradicated.

Scribing.

Scribing in joiners' work is cutting an irregular edge to meet some existing irregularity in another part, or to fit the end of one moulding over another instead of mitreing in—for example, a cupboard front put in after the room is finished may be scribed to the skirting and cornice. The horizontal bars of a sash may be scribed to the vertical bars instead of mitred. The bottom of a skirting may be scribed to the floor when the latter is irregular. The general method of scribing is to take a pair of carpenter's compasses, adjust them to the greatest width to be cut, and screw them up firmly, then pass one point along the irregular surface while the other scratches a parallel outline upon the piece to be cut.

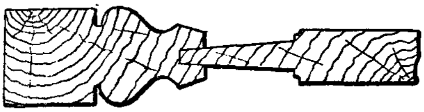


Fig. 798.—Stuck Moulding.

Rebate Defined.

A rebate is a narrow rectangular recess along the edge or across the end of wood, stone, or other material, as in Fig. 794.

Beads.

Stopped Bead.—A stopped bead is one that does not run through to the end of the material.

Bolection Moulding.—A bolection moulding is one that is raised above the surface of the framing, and usually rebated to it. A rebated bolection moulding is shown in Figs. 826 and 827.

Staff Bead.—A staff bead, or angle bead, or double quirked bead, is a bead extending three-quarters of a circle on the edge of any material, and terminated by a quirk on each face, as in Fig. 795.

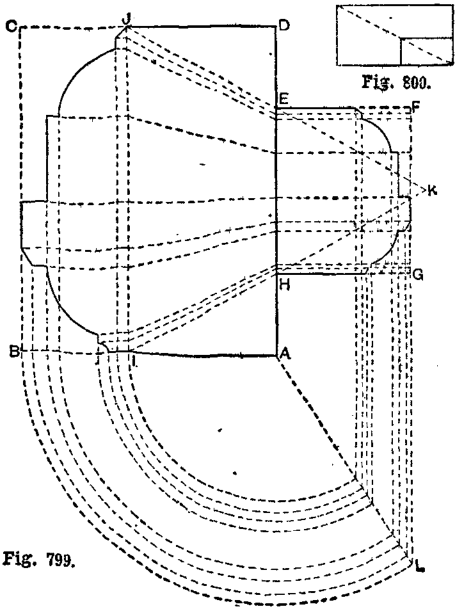


Fig. 800.

Fig. 799.

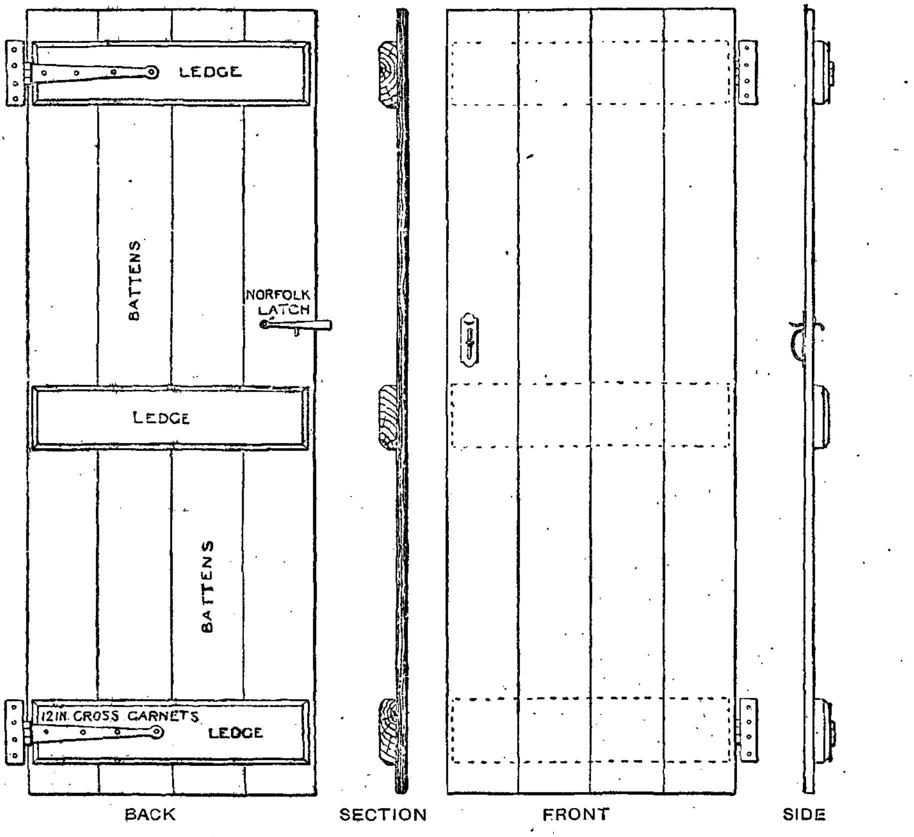
Figs. 799 and 800.—Method of Diminishing Moulding.

Fixing Angle Bead to Wall.

Fig. 796 shows the arrangement of the bead. If wood bricks or coke breeze fixing blocks are built in, the splayed fillets may be nailed to these blocks, otherwise the wall must be plugged as shown, but every alternate plug should be rather nearer the angle. If a plaster quirk be sufficient, the tongued ground may be omitted. If the wall is not plumb the ground must be packed out with wedges or fillets at the back. A plumb rule must be used for testing accuracy.

Mouldings.

Mouldings are of many different sections, according to the class of work and the material. They are used to break the abruptness of the junction between pieces of different thickness, or to ornament the edges. It is not necessary to give many illustrations, as the common sections are well known, and for special work architects design their own mouldings.



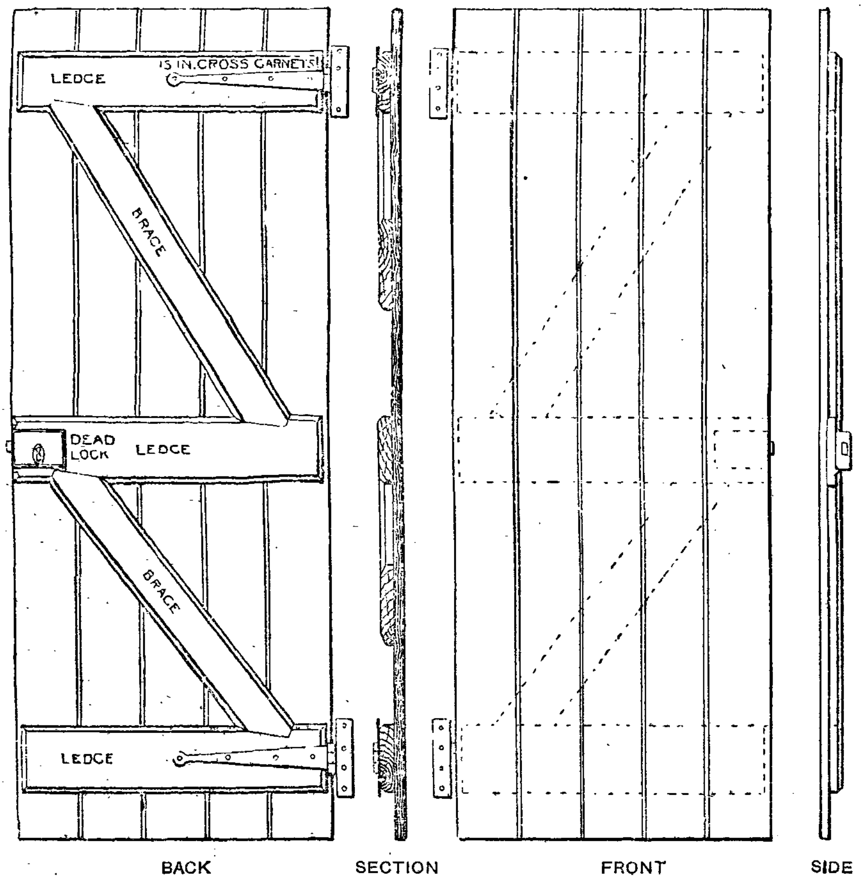
Figs. 801 to 804.—Views of Common Ledged Door.

A planted moulding is one made on a separate piece of stuff and planted or inserted in an angle, as in a framed door (see Fig. 797). A stuck moulding is one worked on the edge of the solid piece, as on a door frame or table edge (see Fig. 798). The stuck moulding is best for all cases when it can be worked along the grain, but in soft wood across the grain, mouldings must be planted on, as at the ends of a bead flush panel.

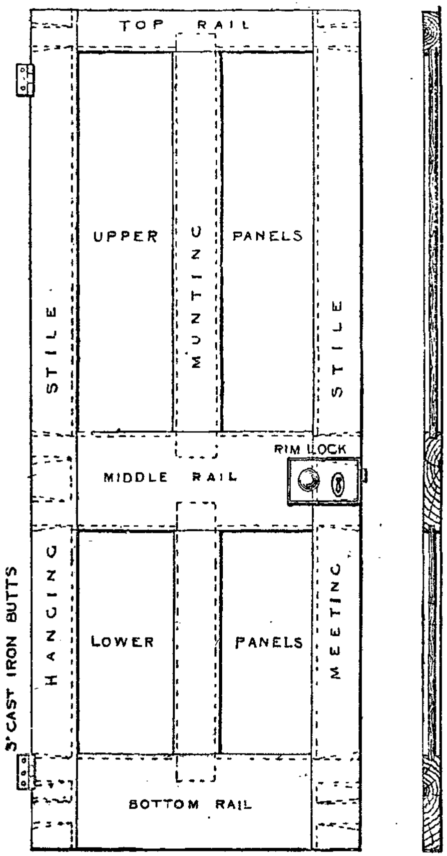
Architrave.—An architrave is the moulding round the sides and top of a door or window, as in Fig. 831; it is also the name given to the lower part of an entablature—the superstructure that lies horizontally upon the columns in classic architecture, the architrave being the part immediately above the capitals of the columns, and surmounted in turn by the frieze and cornice.

Method of Diminishing Mouldings.

The diminishing of mouldings is one of a large class of problems of infinite variety, but all of which are based on the same principles. Let A B C D (Fig. 799) be the section out

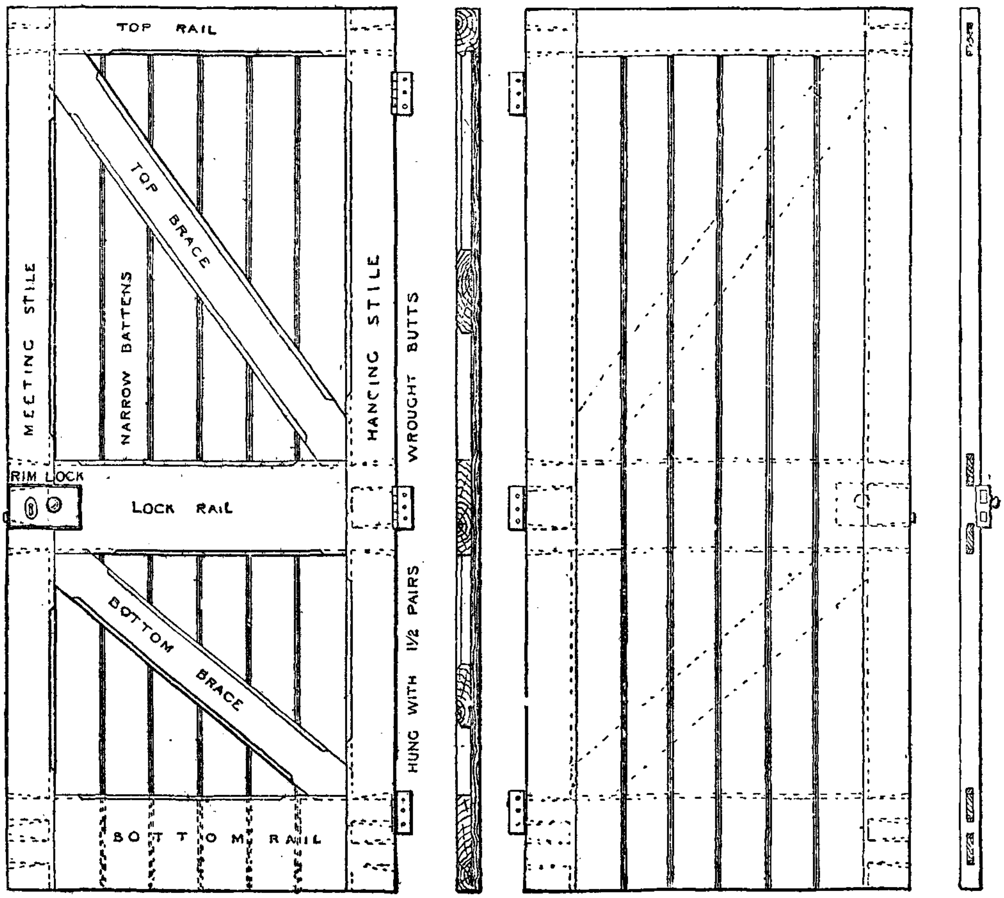

LEDGE
15 IN. CROSS GARNETS
BRACE
DEAD LOCK
LEDGE
BRACE
LEDGE
BACK
SECTION
FRONT
SIDE

Figs. 805 to 808.—Ledged and Braced Door.


TOP RAIL
UPPER
MUNTING
PANELS
STILE
STILE
RIM LOCK
MIDDLE RAIL
HANGING
LOWER
PANELS
MEETING
3" CAST IRON BUTTS
BOTTOM RAIL

Figs. 809 and 810.—1½-in. Square Framed and Flat Four panel Door.

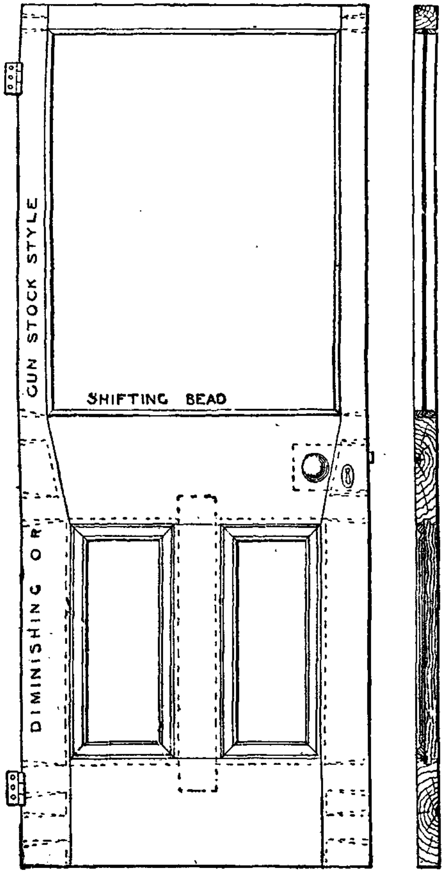
of which the larger moulding is to be cut, and E F G H the section out of which the smaller moulding is to be cut. The mouldings will only be exactly similar when the same diagonal can be produced through the two pieces, as shown in Fig. 800; but the method here shown applies equally well when the pieces of stuff are not in the same proportion, the moulding then being the nearest available proportion. Draw line I J parallel with A D through the nearest point of the moulding, and parallel with this draw other lines from the various angles of the moulding. Also draw lines at right angles to these through the same points, so as to meet the line I J. Now draw lines from the points I J through the corners of the small piece of stuff E H, and produce these lines to meet in point K. Draw lines from all the other intersections on I J to cut line E H, producing these lines parallel to G H, so as to give points for the angles of the new moulding. Then produce F G downwards to meet the curve struck from the centre A with radius A B in point L. Join A L. From centre A with varying radius transfer all intersections on A B to A L, and from the points on A L produce lines parallel with D A and F G to cut the horizontal lines and give intersections for angles of new moulding.



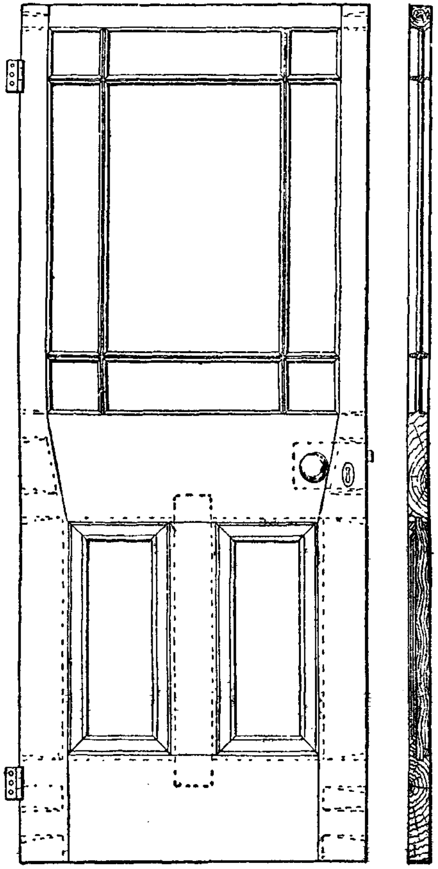
Figs. 811 to 814.—2-in. Framed and Braced Narrow Batten Door.

Woodworking Machinery.

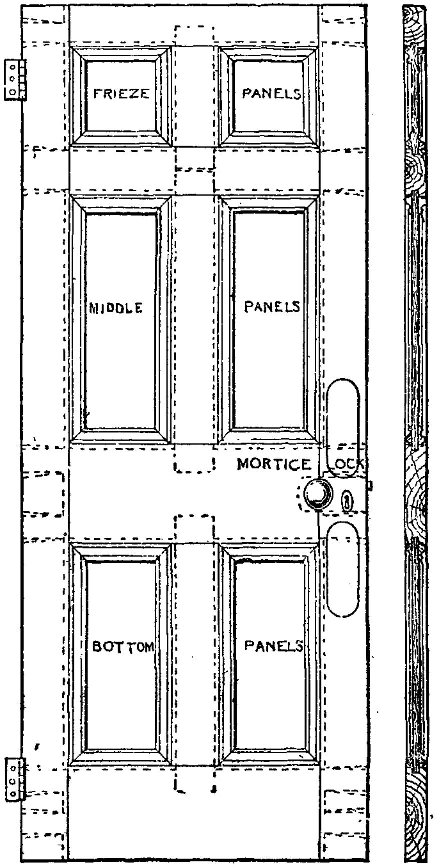
A "general joiner," known also as a "patent general joiner," and as a "variety woodwork-


GUN STOCK STYLE
SHIFTING BEAD
DIMINISHING OR

Figs. 815 and 816.—Sash Door.

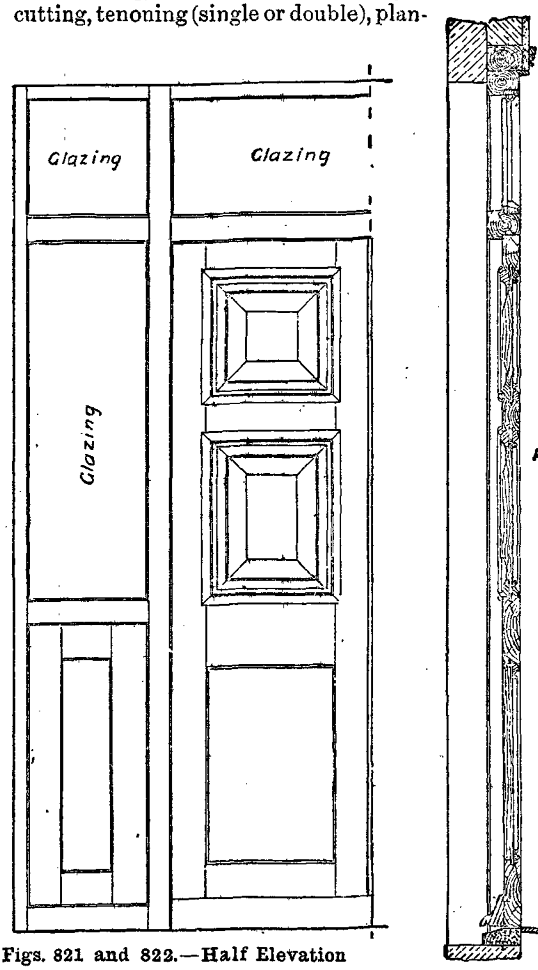


Figs. 817 and 818.—Sash Door with Margins.

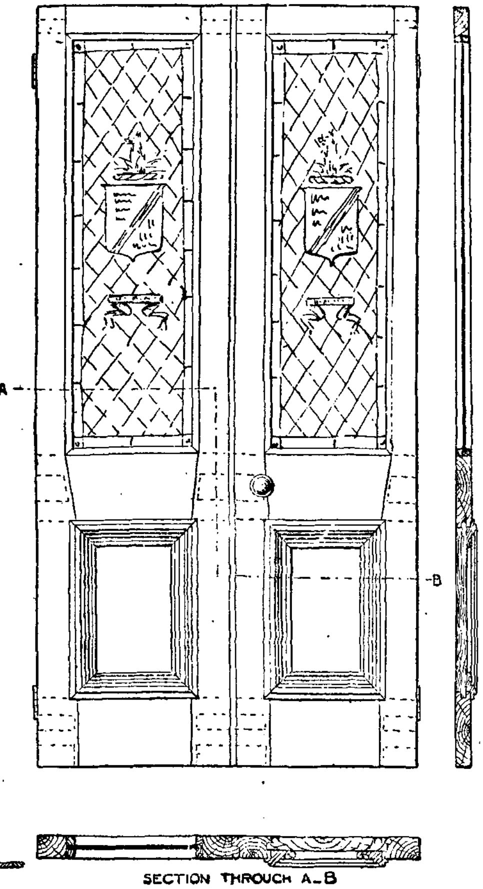

FRIEZE
PANELS
MIDDLE
PANELS
MORTICE
LOCK
BOTTOM
PANELS

Figs. 819 and 820.—2-in. Moulded Six-panel Door.

ing machine," is capable of performing almost all the varieties of work usually done by manual labour in the joiners' shop, such as sawing (with and across the grain), slitting and cross-cutting, mitreing, chamfering, wedge-cutting, tenoning (single or double), planing (straight or taper), moulding (straight or curved), beading, recessing, rebating, grooving, tonguing, and squaring-up; also mortising, boring, and curved and irregular work of various kinds. It takes the place of at least five separate machines—namely, saw-bench, and tenoning, moulding, mortising, and boring machines. Slot mortising is done by traversing a rotating cutter sideways, producing a round-ended slot or mortice of any depth. It is similar in action to the slot-drilling machines in the engineers' workshops. Its advantage over chisel mortising is the reduction of vibration and noise, and the removal of the core. Woodworking machines do not usually come under the head of building construction, but occasional questions are asked in the examinations upon the various tools and machines, and candidates should be acquainted with the more common forms and their uses.



Figs. 821 and 822.—Half Elevation of Double-leaved Panelled Door in Framed and Glazed Partition.



Figs. 823 to 825.—Folding Entrance Doors suitable for Town Mansion.

Height and Width of Internal Doors.

According to Vitruvius: — "For Doric temples, the aperture of the door is determined as follows: The height from the pavement to the lacunaria is to be divided into three parts and a half, of which two constitute the height of the door. The height thus obtained is to be divided into twelve parts, of which five and a half are given to the width of the bottom part of the door. This is diminished towards the top, equal to one-third of the dressing, if the height be not more than 16 ft. From 16 ft. to 25 ft. the upper part of the opening is contracted one-fourth part of the dressing. From 25 ft. to 30 ft. the upper part is contracted one-eighth of the dressing. Those that are higher should have their sides vertical. . . . If the doors are Ionic, their height is to be regulated as in those that are Doric. Their width is found by dividing the height into two parts and a half, and taking one and a half for the width below. The diminution is to be as in the Doric doorway. . . .

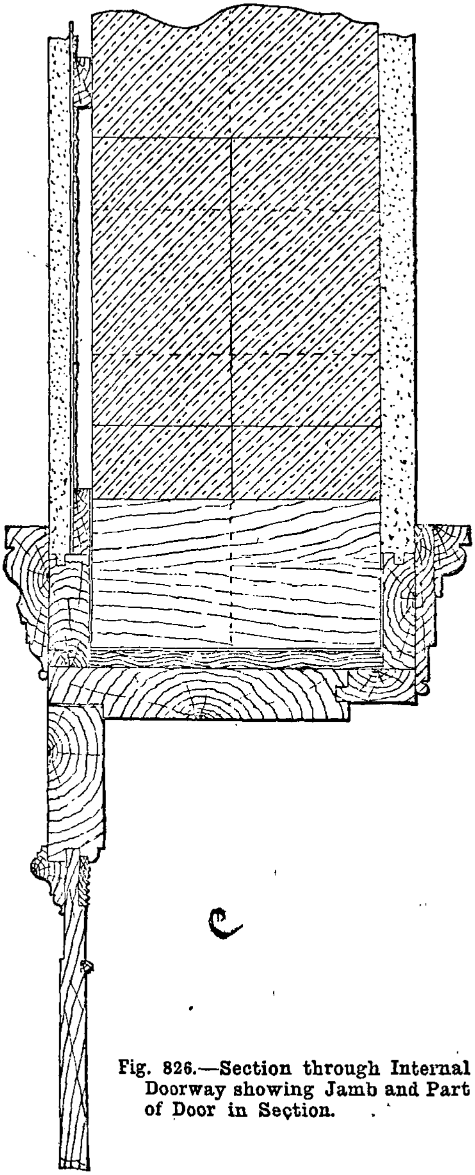


Fig. 826.—Section through Internal Doorway showing Jamb and Part of Door in Section.

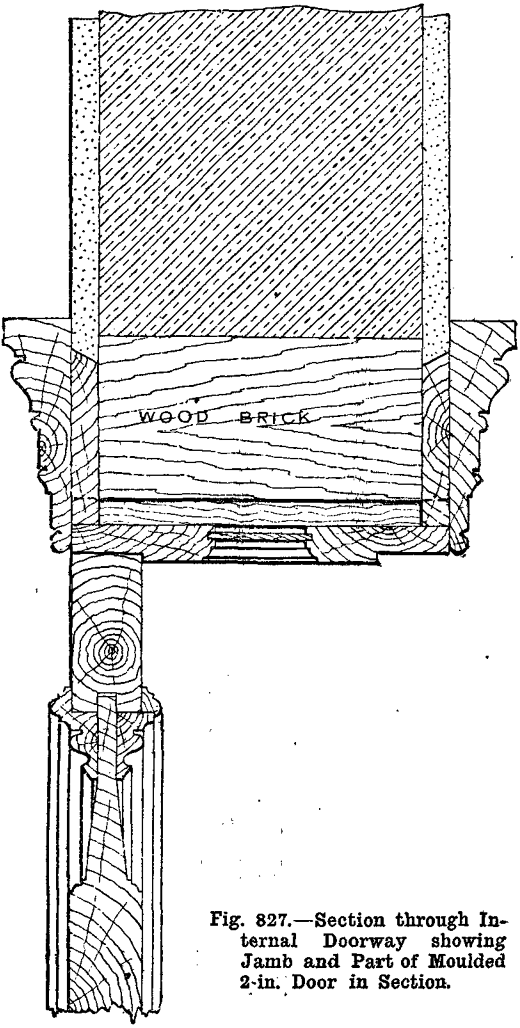


Fig. 827.—Section through Internal Doorway showing Jamb and Part of Moulded 2-in. Door in Section.

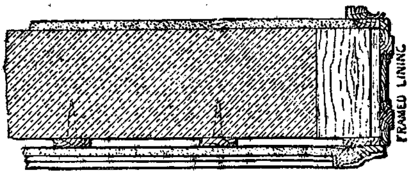

FRAMED LINING

Fig. 828.

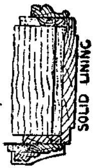

SOLID LINING

Fig. 829

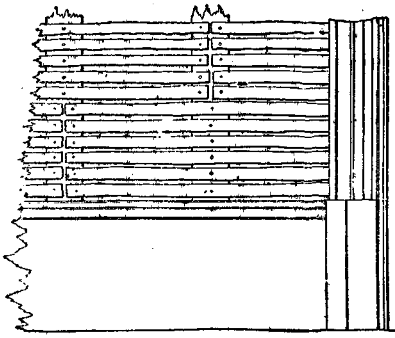


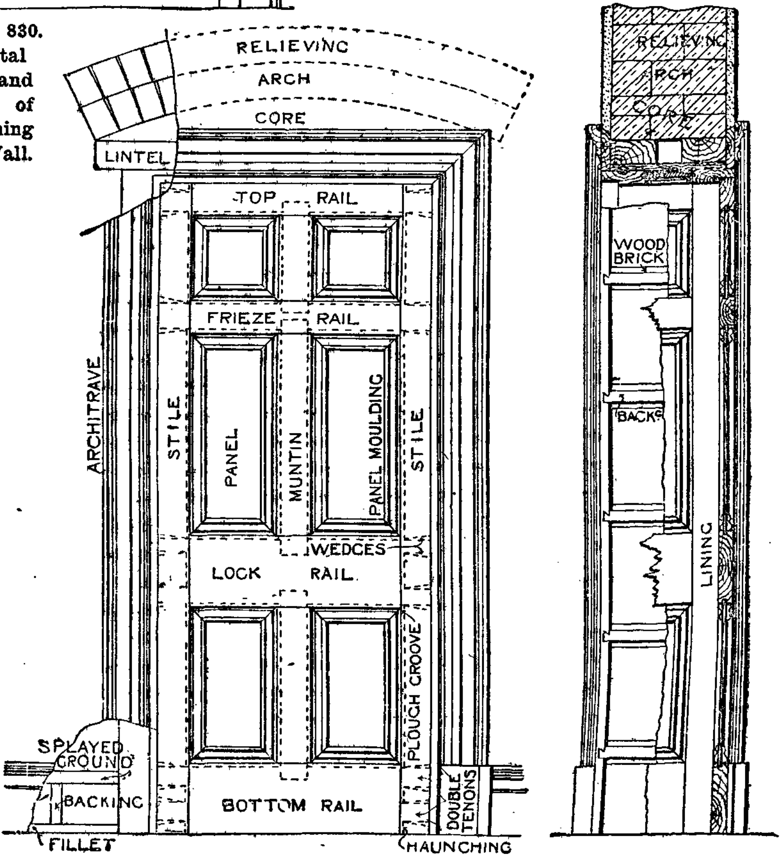
Fig. 830.

Figs. 828 to 830.—Horizontal Sections and Elevation of Door Opening in 9-in. Wall.

If the doors are folding, the height remains the same, but the width is to be increased. "If in four folds, the height is to be increased." For dwelling-houses a good rule for interior doors is: Quarter height of room + 4¼ ft. = height of door; height of door − 4 ft. = width of door. When width of door is given, the ordinary rule is to add 4 ft. for the height.

Varieties of Doors.

The construction of a common ledged door is shown by Figs. 801 to 804; ledged and braced door, Figs. 805 to 808; 1½-in. square framed and flat four-panel door, Figs. 809 and 810; 2-in. framed and braced narrow batten door, Figs. 811 to 814; an ordinary sash door, Figs. 815 and 816; a sash door with margins, Figs. 817 and 818; and a 2-in. moulded six-


RELIEVING
ARCH
CORE
LINTEL
TOP RAIL
FRIEZE RAIL
ARCHITRAVE
STILE
PANEL
MUNTIN
PANEL MOULDING
STILE
WEDGES
LOCK RAIL
PLOUGH GROOVE
SPLAYED GROUND
BACKING
BOTTOM RAIL
DOUBLE TENONS
FILLET
HAUNCHING
RELIEVING
ARCH
CORE
WOOD BRICK
BACKG
LINING

Figs. 831 and 832.—Elevation and Vertical Section of Six panel Door and Doorway.

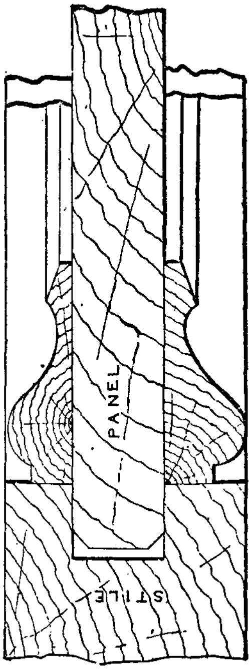

PANEL
STILE

Fig. 833.—Section through Door, showing Stile, Panel and Mouldings.

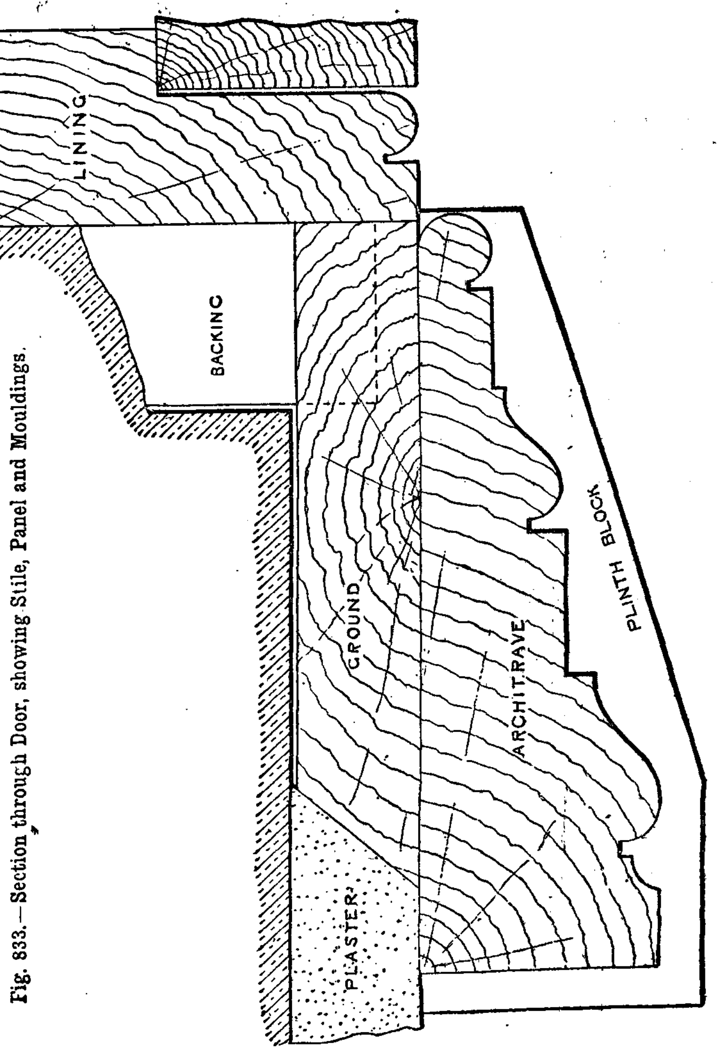

LINING
BACKING
GROUND
ARCHITRAVE
PLINTH BLOCK
PLASTER

Fig. 835.—Horizontal Section through Angle of Doorway.

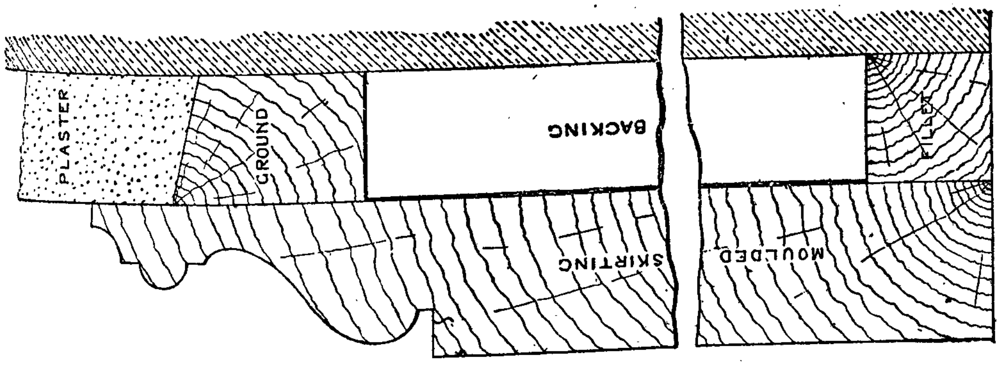

PLASTER
GROUND
BACKING
SKIRTING
MOULDED
FILLET

Fig. 834.—Section through Moulded Skirtings.

panel door, Figs. 819 and 820; Fig. 821 is the half-outside elevation of a double-leaved hall or porch door. The door opening is 7 ft. 3 in. ×4 ft.; each leaf of the door is in three panels; the top panels have raised centres, bolection moulded; lowest panels are flush and beaded. A top light and sidelights are shown. The opening is as follows: Soffit of reveal to top of door sill 9 ft.; from reveal to reveal, 7 ft. 6 in. Fig. 822 is a vertical section from 6 in. above soffit to doorstep, showing door frame and sash of top light, and passing through panels of door, showing weather board and door sill. Figs. 823 to 825 show elevation and section of a pair of folding doors for an entrance doorway, 4 ft. by 8 ft., to a town mansion.

Doorway Details.

Fig. 826, on p. 201, shows a horizontal section through one jamb of an internal doorway in a first-class dwelling-house. The walls are of 9-in. brickwork, rendered in passage, and lathed and plastered in room. All the details of the wood fittings are shown, including a part of the door, the section of which is through the panels. Fig. 827, on p. 201, shows a horizontal section through the door jamb of an internal door in a 9-in. wall of a first-class dwelling-house, with full details, including one stile and part of the panel of a handsomely moulded 2-in. door. In fixing the jamb linings, etc., to a doorway in a 9-in. wall, the first step is to fix the grounds, which are secured to wood bricks, pallets, or coke breeze fixing blocks built into the wall. The grounds should be wrought both sides, framed, and bevelled on the back edges to form a key for the plaster. Their width is governed by that of the architrave, which should overlap the ground ¾ in. The two sets of grounds on each side of the opening should be connected across the face of the jambs with 1½-in. by 1-in. backing pieces dovetailed in, and arranged to come behind the rails of the linings. The linings are then fixed to the grounds, and the hinges of the door are fixed in the rebate. The grounds and ends of the linings are covered by an architrave moulding. The grounds must be packed out if the brickwork is not truly built. If the door is hung with rising butts the top of the door and head rebate should be slightly bevelled outwards to allow it to open freely. Fig. 828 shows a horizontal section through part of an internal doorway opening in a 9-in. brick wall. There is a double rebated and beaded lining secured to ¾-in. grounds. On one side of the wall rendering and a single moulded architrave are shown, and on the other side lath and

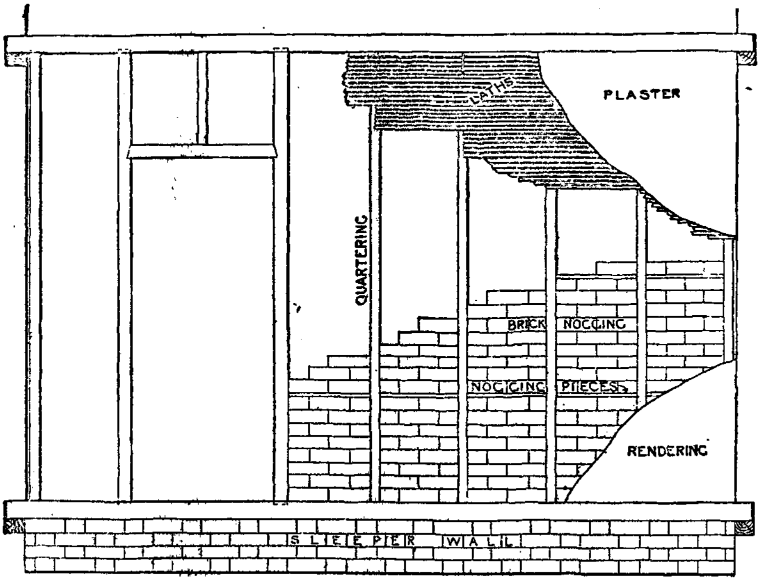


Fig. 836.—Common Stud Partition.

plaster on battens, and a double-faced architrave. A framed lining is shown in Fig. 828, and a solid lining in Fig. 829. The elevation (Fig. 830) shows the naked lathing on battens, the architrave with a plinth, and a moulded skirting 11 in. high.

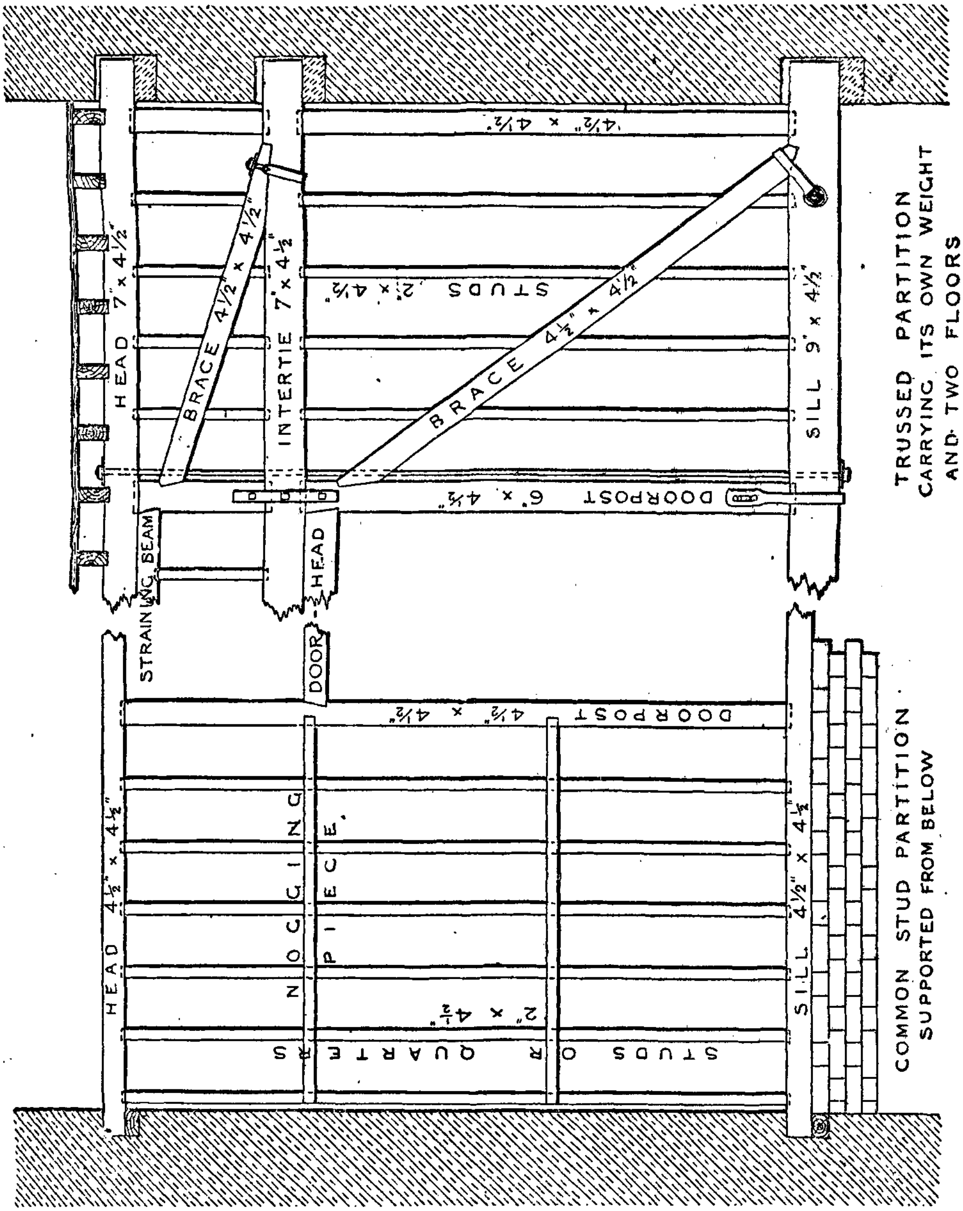

HEAD 7" × 4½"
BRACE 4½" × 4½"
INTERTIE 7" × 4½"
4½" × 4½"
STUDS 2" × 4½"
BRACE 4½" × 4½"
SILL 9" × 4½"
DOORPOST 6" × 4½"
STRAINING BEAM
HEAD
DOOR
DOORPOST 4½" × 4½"
HEAD 4½" × 4½"
NOGGING
PIECE
SILL 4½" × 4½"
2" × 4½"
STUDS OR QUARTERS
TRUSSED PARTITION
CARRYING ITS OWN WEIGHT
AND TWO FLOORS
COMMON STUD PARTITION
SUPPORTED FROM BELOW

Fig. 837.—Halves of Common Stud and Trussed Partitions.

Six-panel Door and Opening.

Figs. 831 and 832 show respectively front elevation and section of a 2-in. six-panel door to fill an opening 7 ft. by 3 ft. in an internal 14-in. wall. The figures, together with the detail views (Figs. 833 to 835), show the method

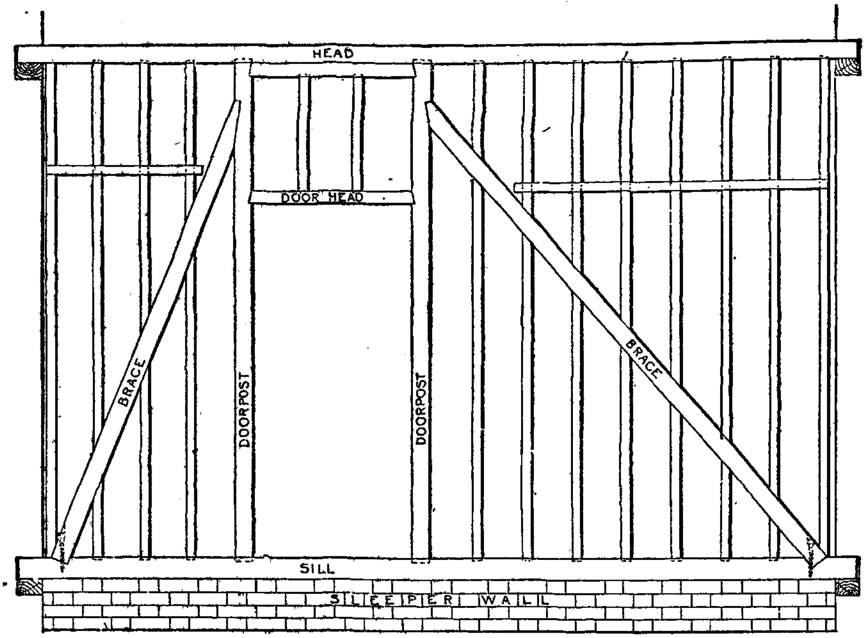

HEAD
DOOR HEAD
BRACE
DOORPOST
DOORPOST
BRACE
SILL
SLEEPER WALL

Fig. 838.—Braced Partition.

of framing, also the grounds, jamb linings, and architraves, and give the technical names of the different parts.

Partitions, Stud, Braced and Trussed. A common stud partition is shown by Fig. 836; and in Fig. 837 is shown half each of a

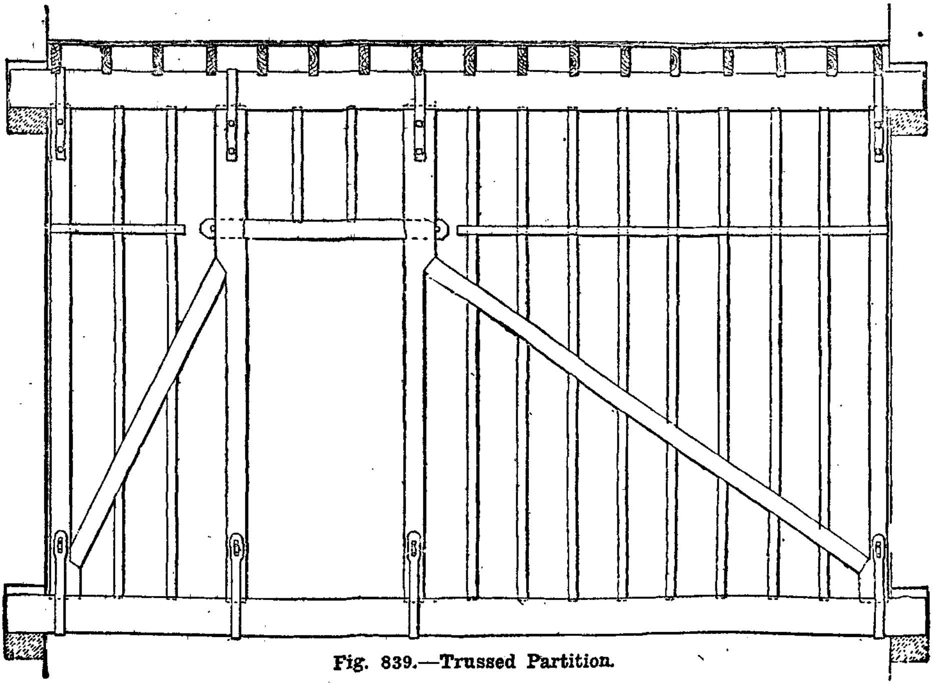


Fig. 839.—Trussed Partition.

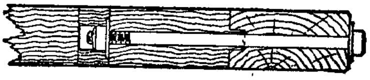


Fig. 841.—Section of Foot of Post showing Bolt.

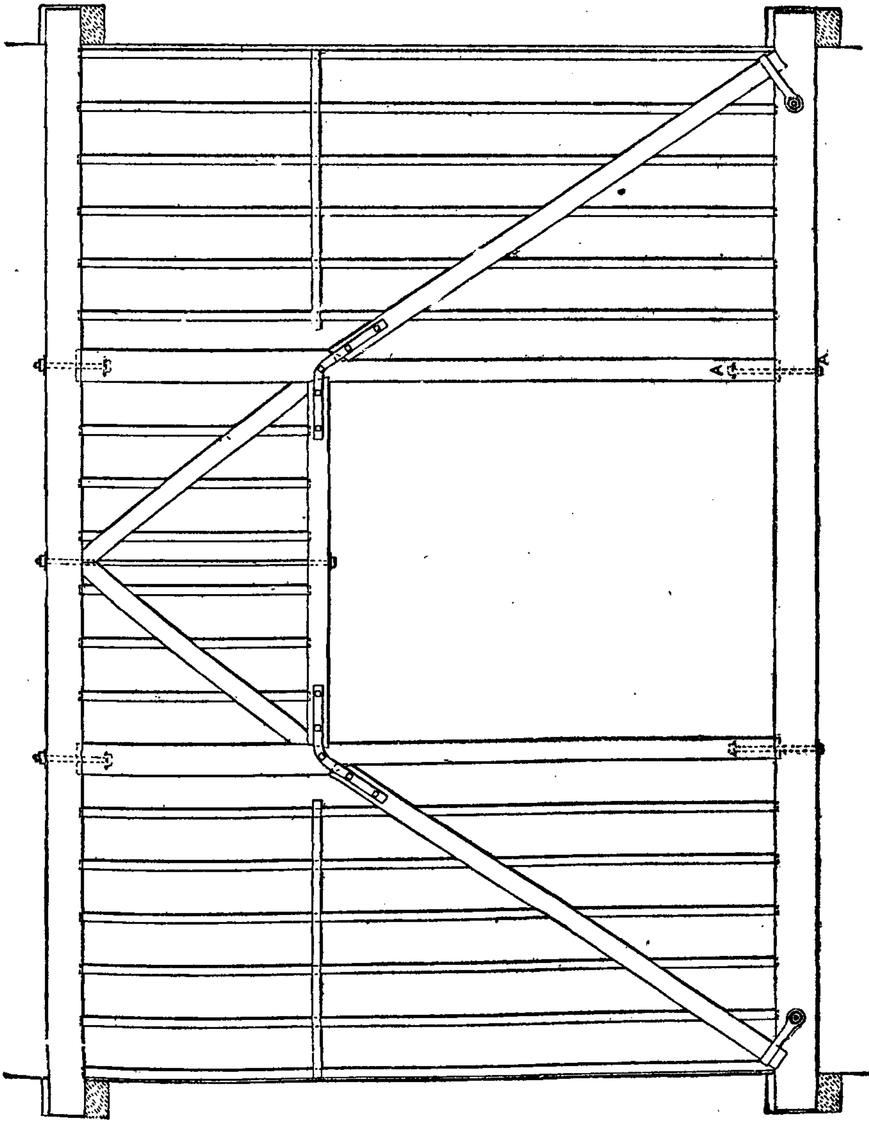


Fig. 840.—Trussed Partition with Opening for Folding Doors.

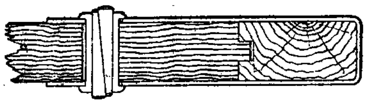


Fig. 843.—Section of Foot of Post, showing Iron Strap, Cotters, etc.

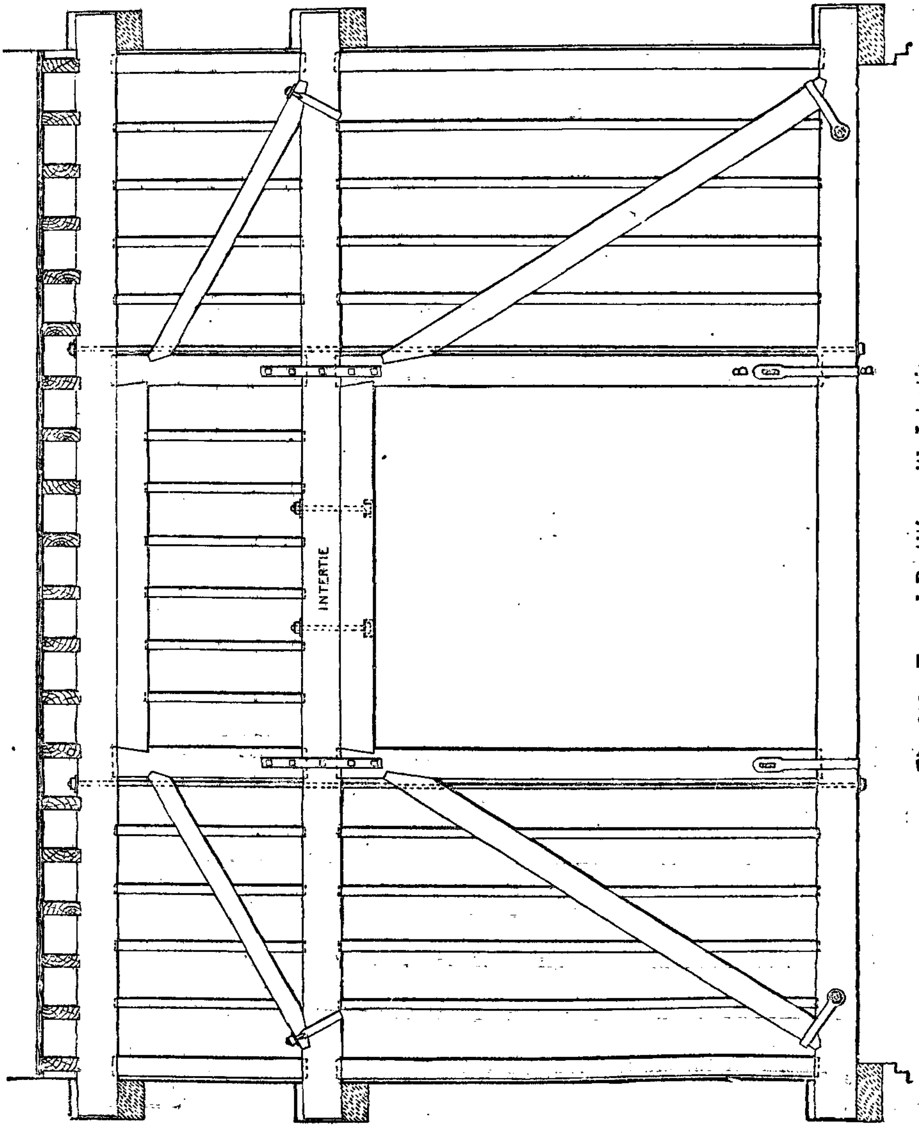


Fig. 842.—Trussed Partition with Intertie.

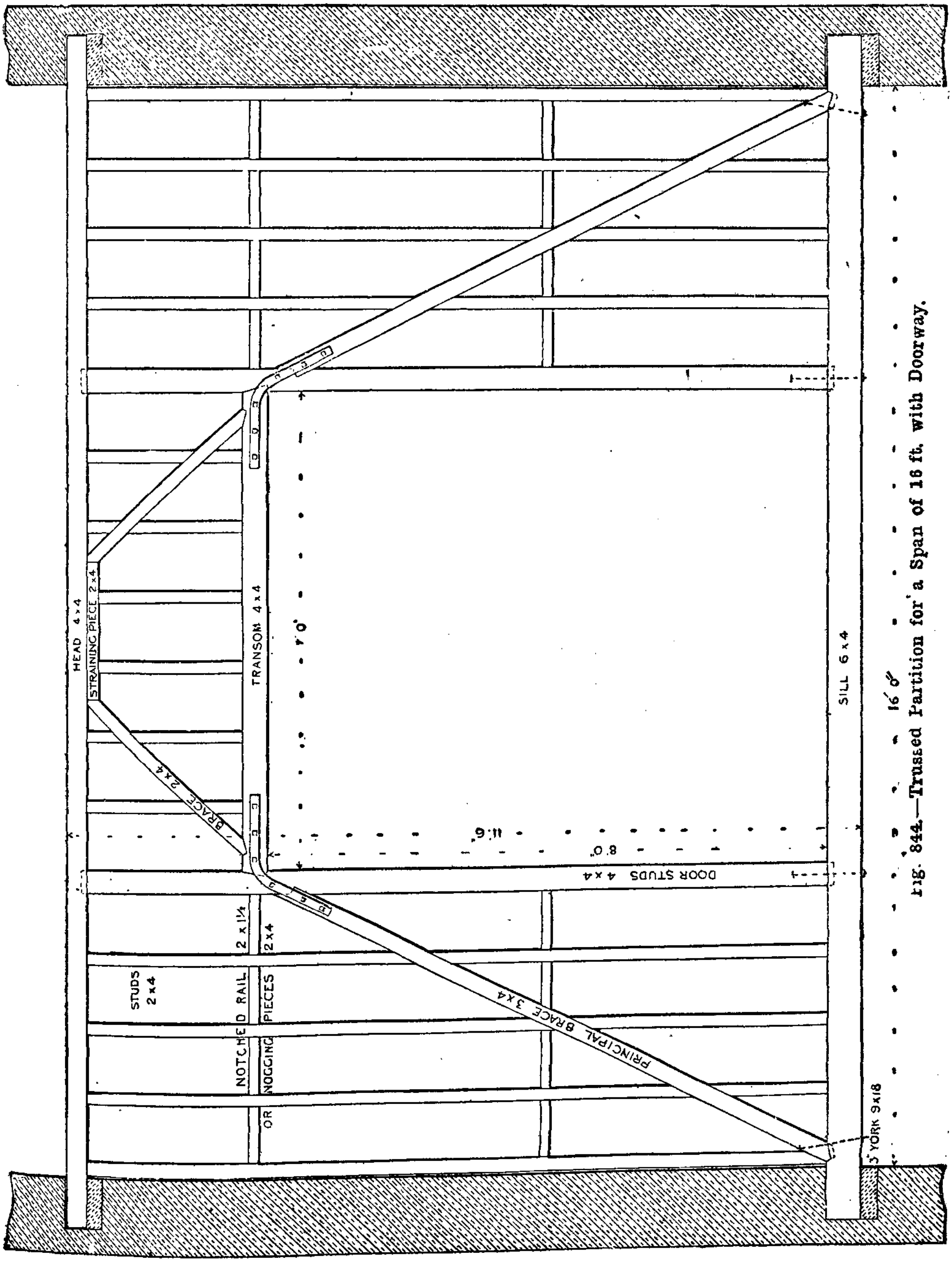

HEAD 4×4
STRAINING PIECE 2×4
TRANSOM 4×4
BRACE 2×4
PRINCIPAL BRACE 3×4
DOOR STUDS 4×4
SILL 6×4
STUDS 2×4
NOTCHED RAIL 2×1¼
OR
NOGGING PIECES 2×4
7'0"
11'6"
8'0"
16'0"
3" YORK 9×18

Fig. 844.—Trussed Partition for a Span of 16 ft. with Doorway.

similar partition, which, of course, is supported from below, and of a trussed partition constructed to carry its own weight besides that of two floors. The names and dimensions of the several members are clearly shown. Studs are upright pieces of wood, say 3 in. by 2 in., in a partition to which to attach the laths for plastering. Any short upright piece of wood filled in to a roof, partition, or bridge is often called a stud. A bolt with a thread at each end, one end being screwed into a casting and the other having a nut, to hold a keep or cover, is also called a stud. An ordinary braced partition is illustrated by Fig. 838; a form of trussed partition by Fig. 839; a trussed partition with opening for folding doors by Fig. 840; an enlarged section at A A being shown by Fig. 841, and a trussed partition with an intertie by Fig. 842, an enlarged section at B B being shown by Fig. 843. The trussed partition shown by Fig. 844 is 16 ft. span and 11 ft. 6 in. high, and it has a central opening for a doorway 7 ft. wide. Fig. 845 is the half elevation of a trussed partition carrying a floor above.

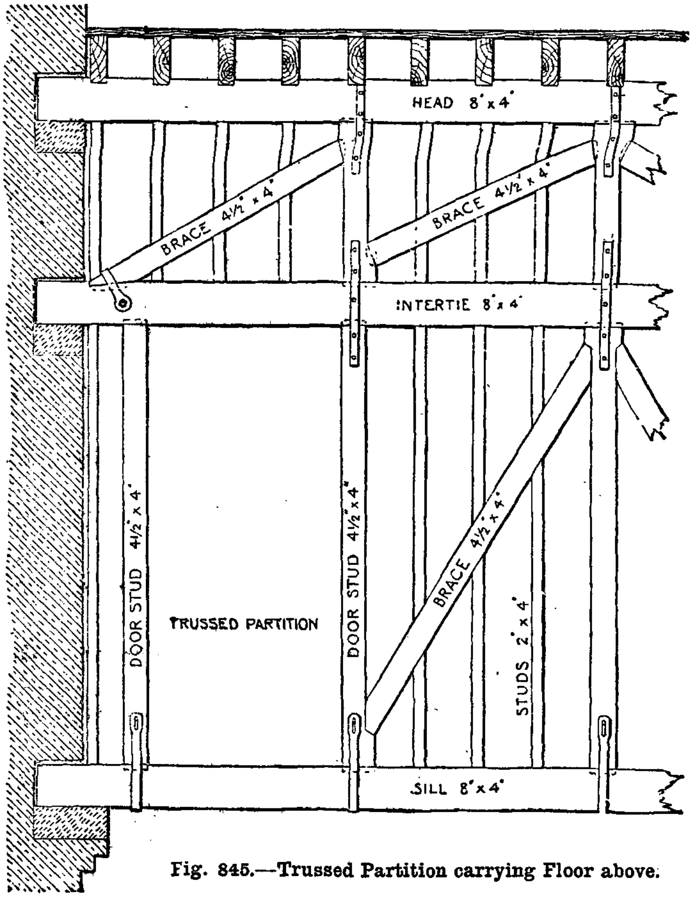
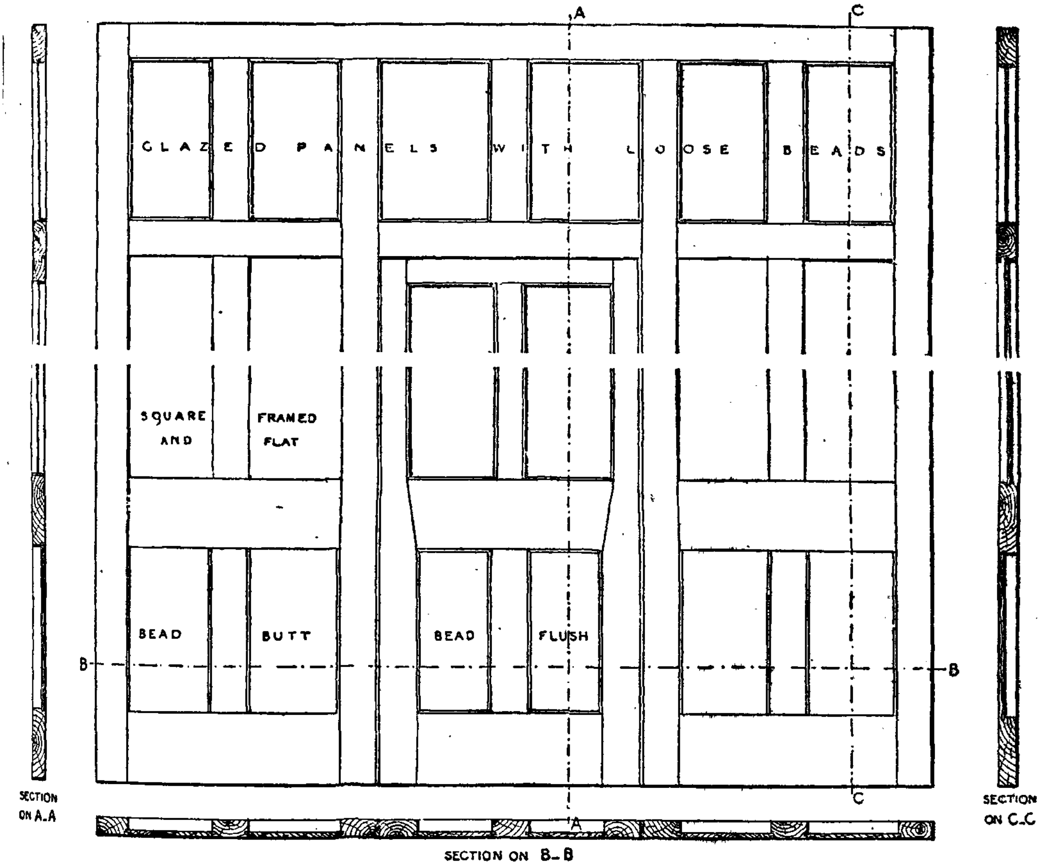


Fig. 845.—Trussed Partition carrying Floor above.

Framed Partitions.

A 2¼-in. framed partition is illustrated by Figs. 846 to 848; a 2-in. moulded partition by Fig. 849; an oak moulded partition is shown in elevation by Fig. 850, various sections being represented by Figs. 851 to 855; a 1½-in. dwarf partition with gate is shown by Fig. 856; and a 1½-in. office enclosure is shown by Figs. 857 and 858, a cast-iron knee being shown in section by Fig. 859. The 16-ft. by 11½-ft. framed partition shown in part by Fig. 860 is designed to assist


A
C
GLAZED PANELS WITH LOOSE BEADS
SQUARE AND FRAMED FLAT
BEAD BUTT
BEAD FLUSH
B
B
A
C
SECTION ON A.A
SECTION ON C.C
SECTION ON B-B

Figs. 846 to 848.—2¼-in. Framed Partition.

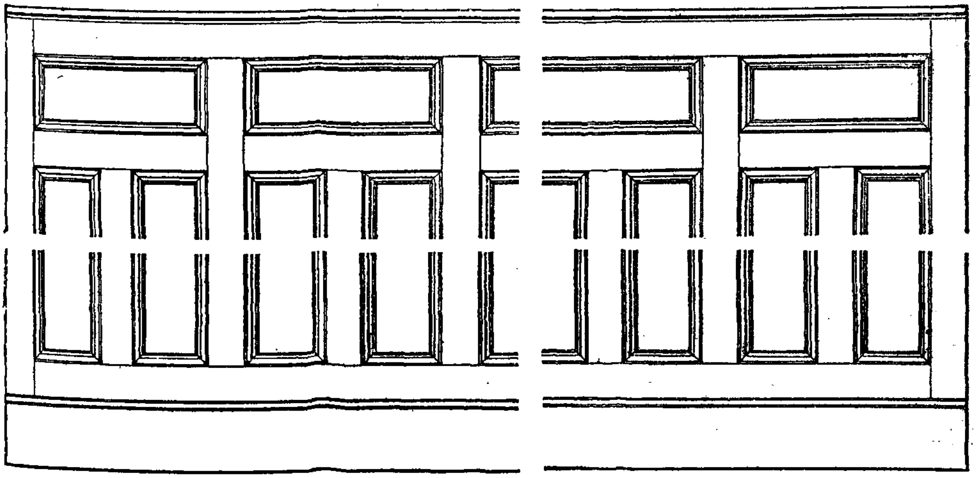


Fig. 849.—2-in. Moulded Partition.

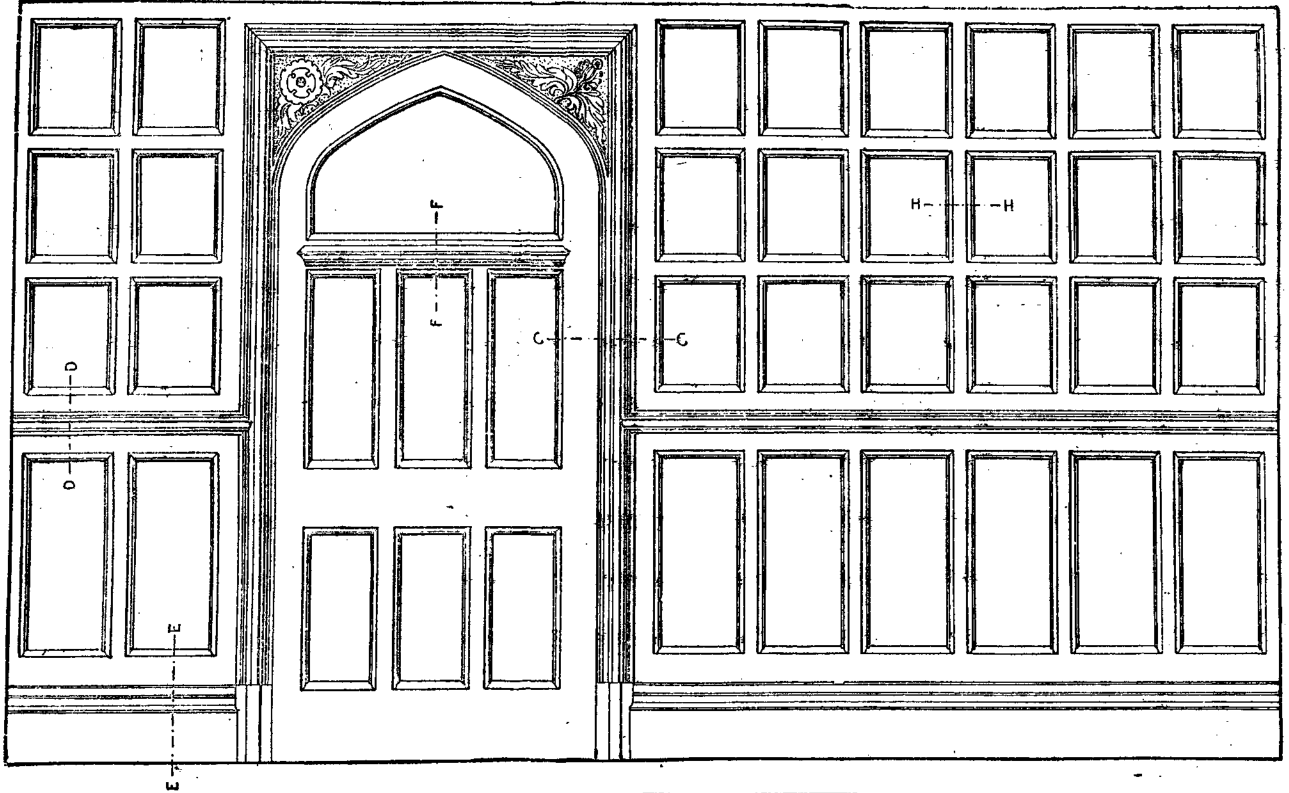
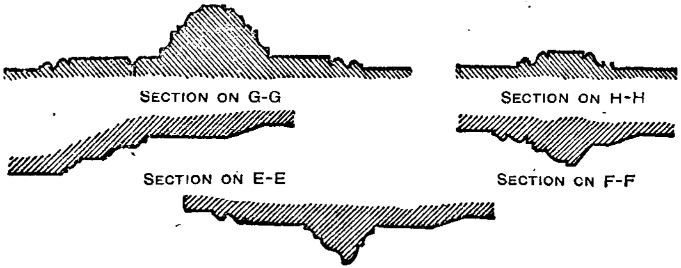


Fig. 850.—Oak Moulded Partition.


SECTION ON G-G

SECTION ON H-H

SECTION ON E-E

SECTION ON F-F

SECTION ON D-D

Figs. 851 to 855.—Sections through Mouldings of Partition.

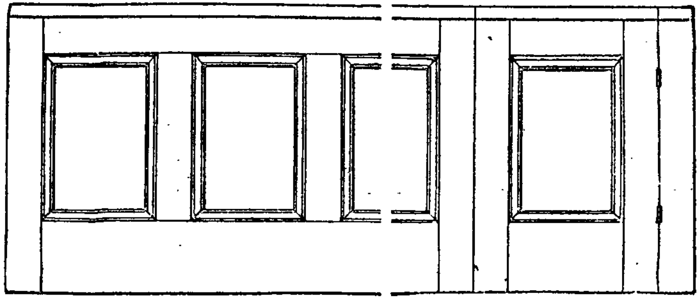
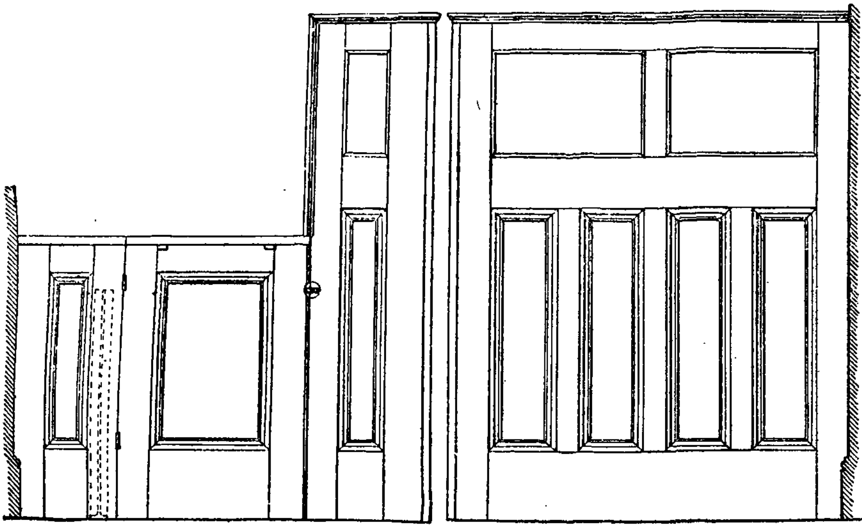


Fig. 856.—1½-in. Dwarf Partition with Gate.



Figs. 857 and 858.—1½-in. Office Enclosure and Counter.

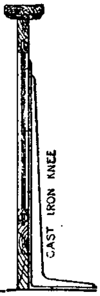

CAST IRON KNEE

Fig. 859.—Cast-iron Knee.

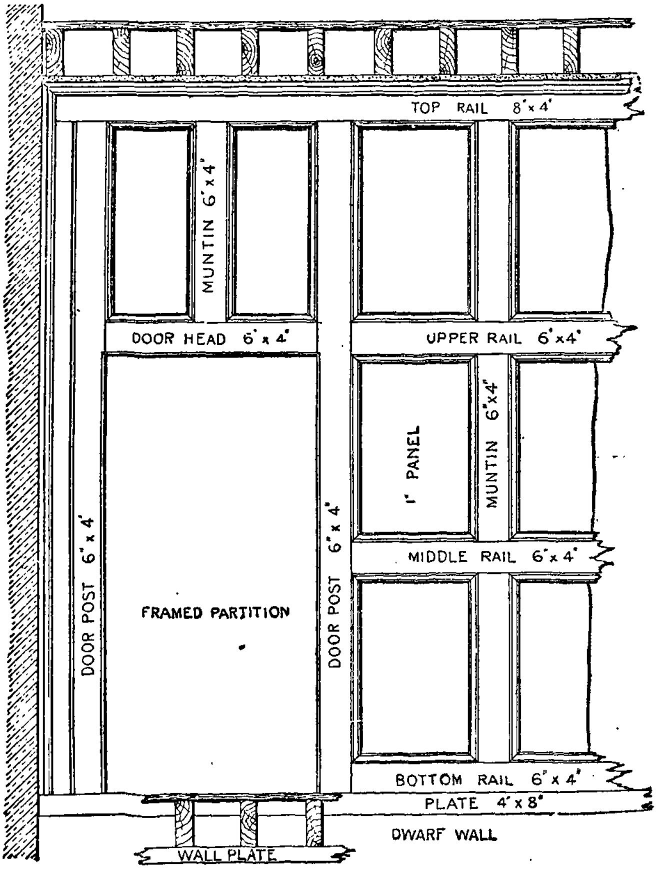

TOP RAIL 8"x 4"
MUNTIN 6"x 4"
DOOR HEAD 6" x 4"
UPPER RAIL 6"x4"
1" PANEL
MUNTIN 6"x4"
DOOR POST 6" x 4"
FRAMED PARTITION
DOOR POST 6" x 4"
MIDDLE RAIL 6"x 4"
BOTTOM RAIL 6" x 4"
PLATE 4"x 8"
DWARF WALL
WALL PLATE

Fig. 860.—Partition carrying Floor above.

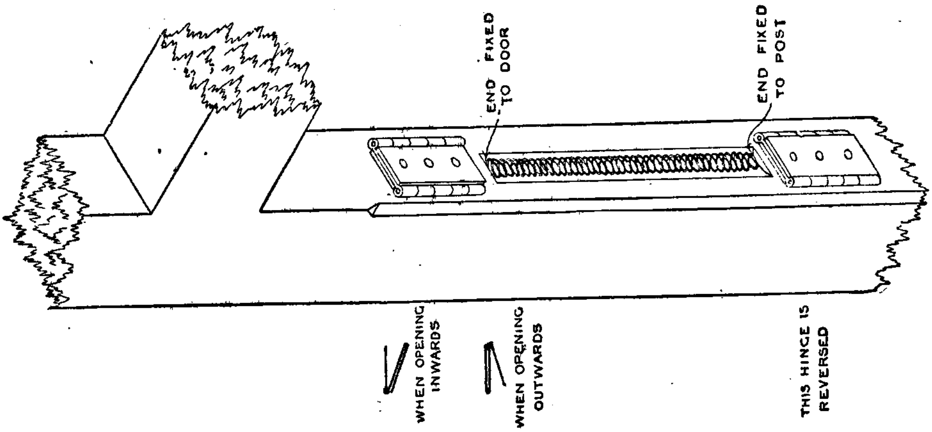
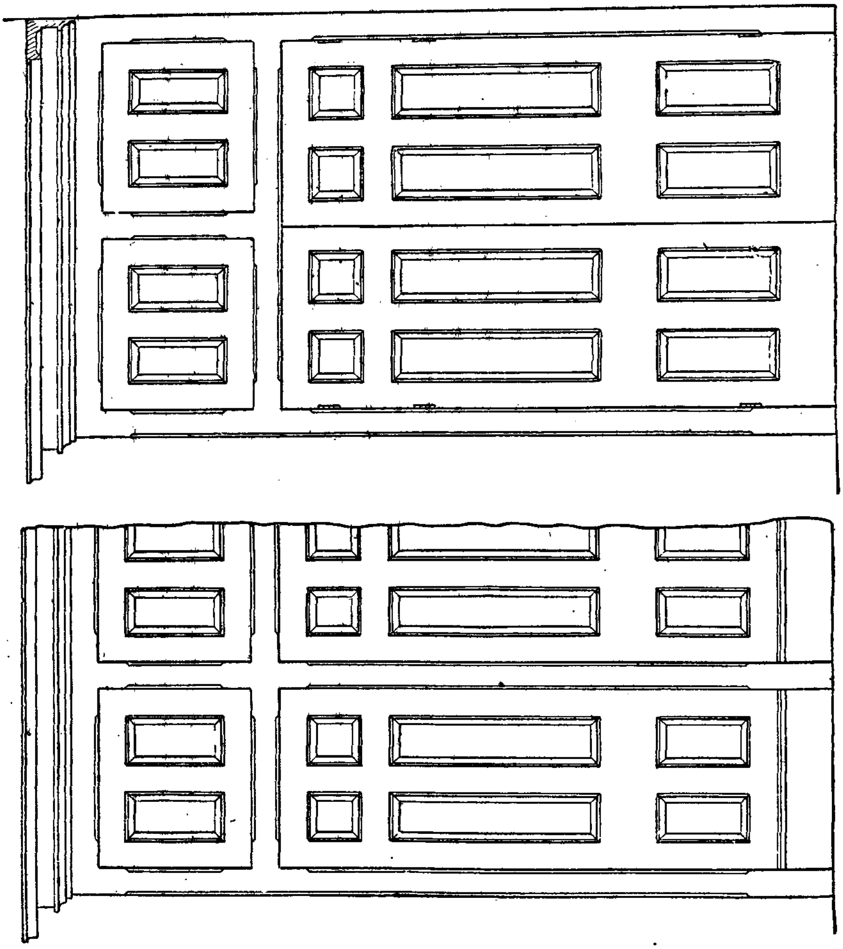

END FIXED TO DOOR
END FIXED TO POST
WHEN OPENING INWARDS
WHEN OPENING OUTWARDS
THIS HINGE IS REVERSED

Fig. 863.—Arrangement for Hanging "Spring Wing Doors Hinged to Post."



Figs. 861 and 862.—Pair of Doors and Panelled Partition to Match.

in carrying a floor above; there is a doorway at 1 ft. from each end. The half of a framed enclosure, or vestibule for public hall, with openings filled with panels of framing 2 in. thick, and having panels 1 in. thick, is shown by Fig. 861; a suitable pair of doors being shown in elevation by Fig. 862. The height to ceiling is 11 ft., and the cornice, which is carried round the partition, is 8 in. deep. The arrangement for hanging "spring wing doors hinged to posts" (almost a contradiction in terms) is shown in Fig. 863, three leaved hinges being fixed with a spiral spring sunk into the post between them.

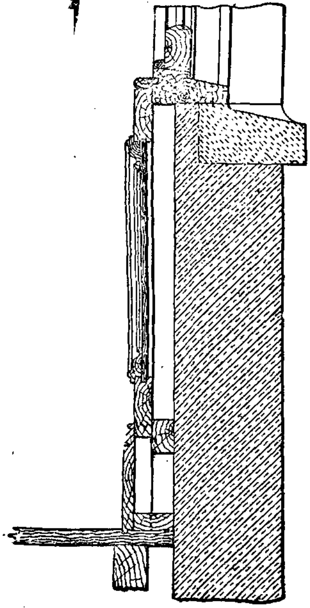


Fig. 864.—Vertical Section through Window Back, 9-in. Brick Wall, etc.

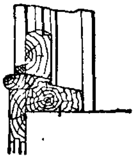


Fig. 865.—Vertical Section showing 5-in. by 3-in. Oak Sill.

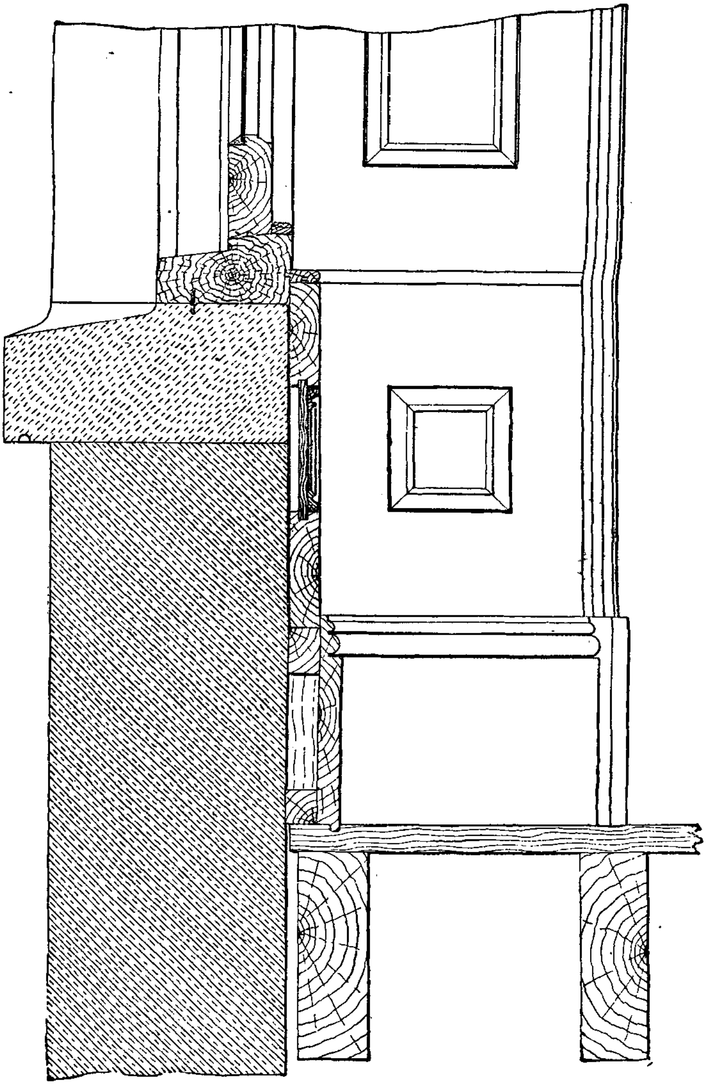


Fig. 866.—Window in Cross Section, showing Bottom Rail of Sash, Wood Sill of Frame, Stone Sill, and Window Back.

Rules for Sizes of Windows.

The following rules are given by different authorities: Area of window surface =

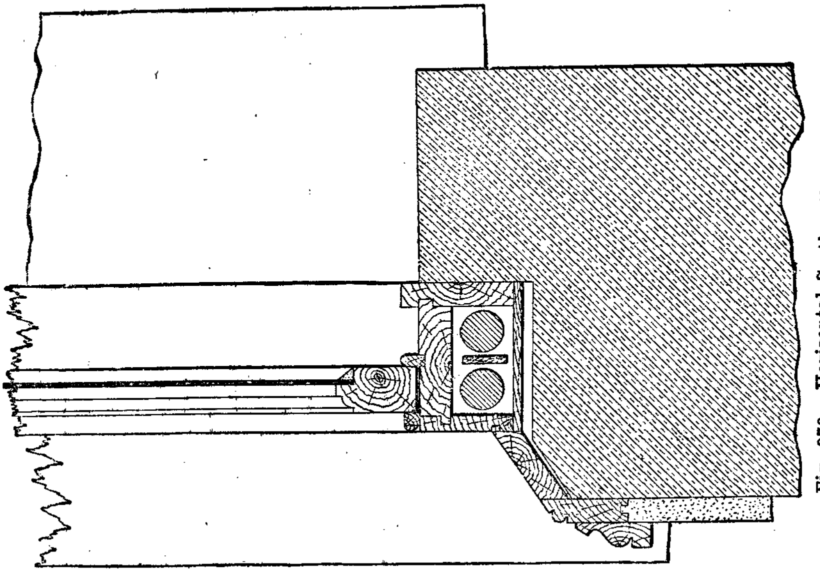
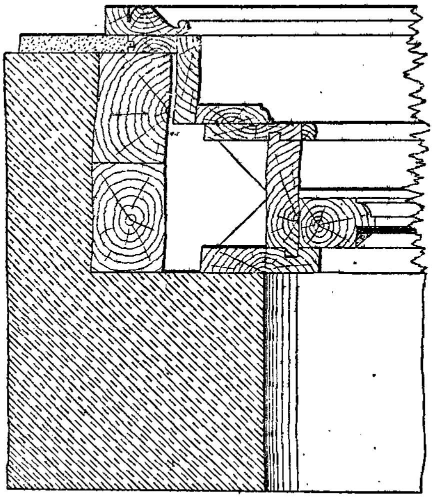


Fig. 870.—Horizontal Section through Frame for Double-hung Sashes.



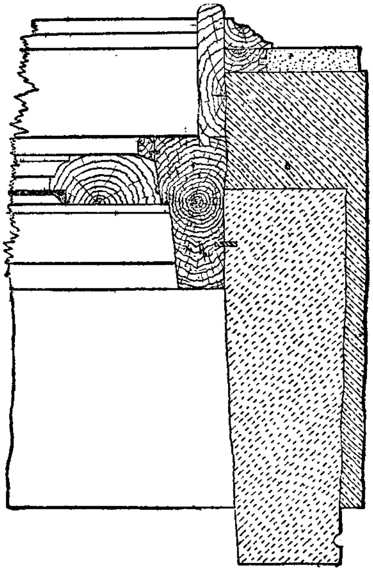


Fig. 868.—Vertical Section through Window Head.
Fig. 869.—Vertical Section through Window Sill.

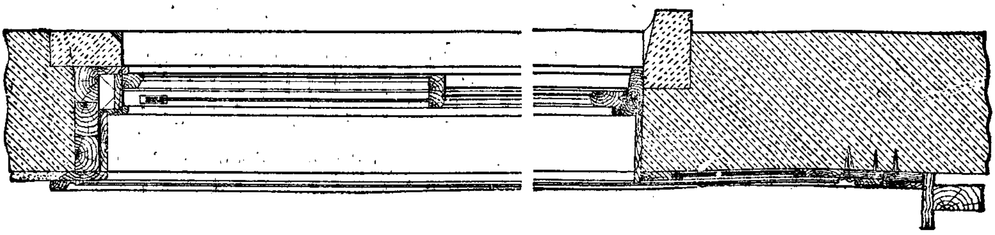


Fig. 867.—Vertical Section through Window Opening with Panelled Window Back.

Fig. 871.—Horizontal Section through Two-Light Window in Stone Wall. Fig. 872.—Section of Cased Frame with Double-hung Sashes. Figs. 873 and 874.—Sections of Window Jamb, Cased Frame, etc., in 18-in. Brick Wall.

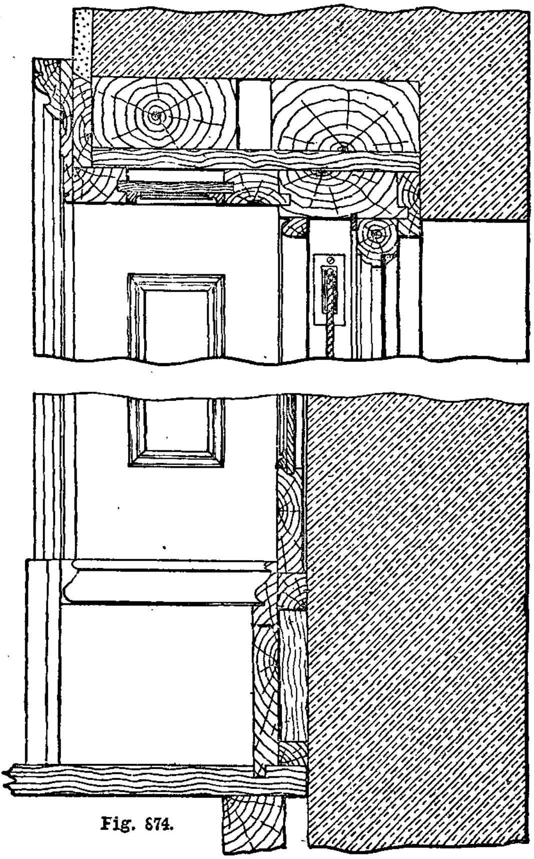


Fig. 874.

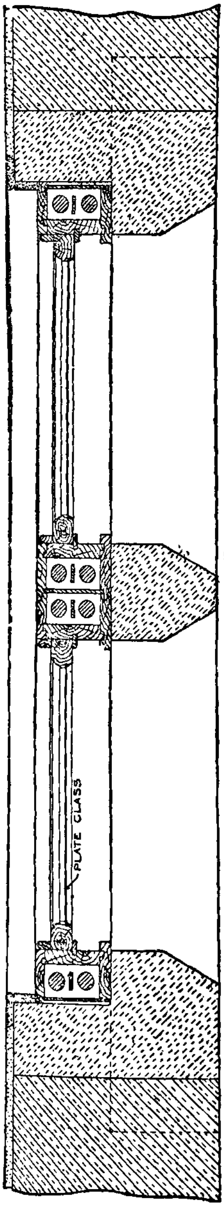


PLATE GLASS

Fig. 871.

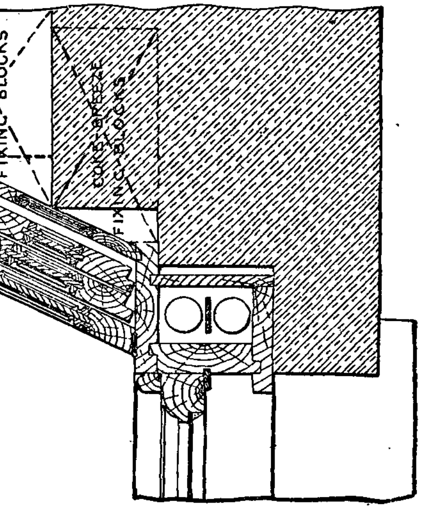

COKE - BREEZE
FIXING BLOCKS
COKE - BREEZE
FIXING BLOCKS

Fig. 873.

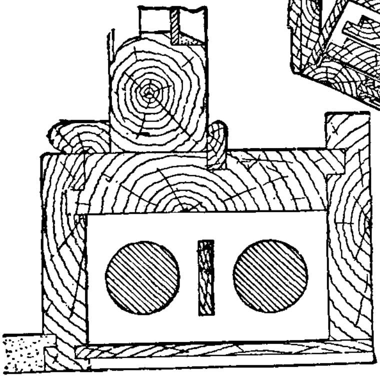


Fig. 872.

$\sqrt{\text{cubic contents of room}}$ (Robert Morris); breadth of window = ⅛ (breadth + height) of room, height of window two to two and a half times breadth (Sir W. Chambers); 1 ft. super. of window space to every 100 cub. ft. or 125 cub. ft. contents of room in dwelling-houses; 1 ft. super. to 50 cub. ft. or 55 cub. ft. in hospitals (Sir Douglas Galton); 1 ft. super. of light to every 100 cub. ft. contents of room (Joseph Gwilt); width of window equals side of square whose diagonal is the height (J. S. Adams). The window sill should be 18 in. to 36 in. above the floor, but the average should be 30 in.; the head of the window should be as high as possible.

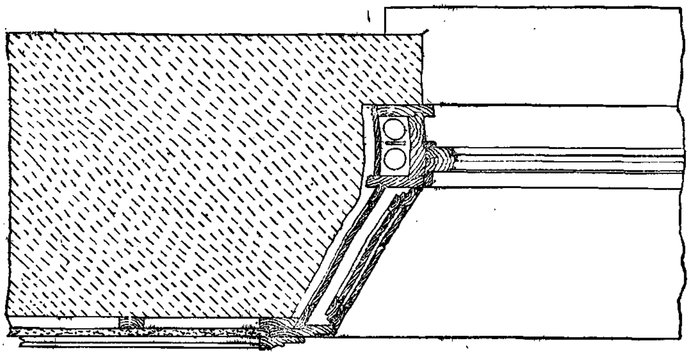


Fig. 875.—Horizontal Section through Window Opening in Stone Wall.

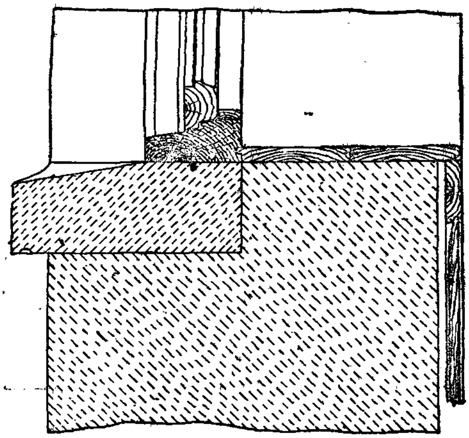


Fig. 876.—Vertical Section through Stone Sill, etc.

Window Backs and Openings.

Fig. 864 is a vertical section through a window back, showing a 9-in. by 5-in. stone sill resting on 9-in. brickwork, a 7-in. by 3-in. oak sill, a 1½-in. bolection-moulded panelled window back lining, also the bottom rail of a 2¼-in. double-hung sash. A 5-in. by 3-in. oak sill is shown in the alternative arrangement (Fig. 865). A cross-section showing bottom

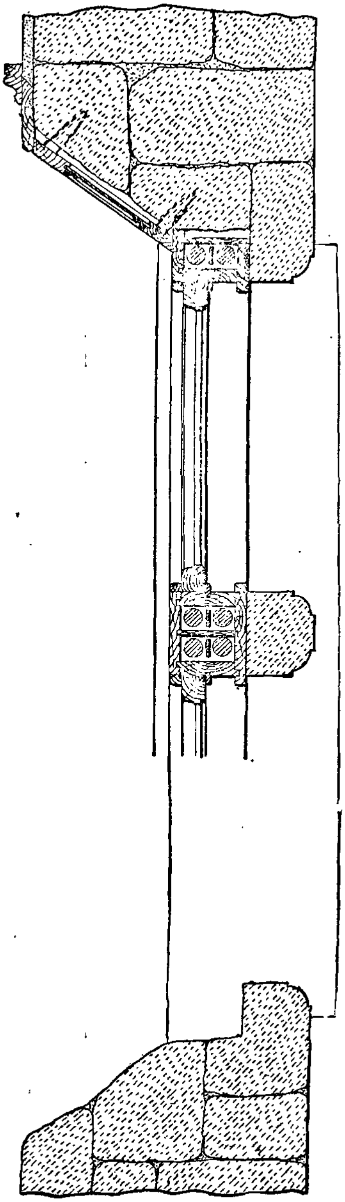


Fig. 877. — Horizontal Section through Two-Light Window in Stone Wall.

rail of sash, wood sill of window frame, stone sill, and window back is given by Fig. 866. The window back is 2 ft. high, and the wall of the recess is 10 in. thick. The window back is continued on the jambs or sides of the recess as shown. Fig. 867 presents a vertical section through a 3-ft. by 7-ft. window opening in an 18-in. brick wall, showing double-hung sashes, stone head and sill, and a panelled window back. Figs. 868 to 870 show a window opening in an 18-in. wall with 9-in. reveal, which is filled with 2-in. double-hung ovolo sashes in deal-cased frame and oak sunk and weathered sill, and finished internally with splayed linings, grooved grounds to stop plastering, and moulded architraves. The jamb, soffit and sill are shown in detail. Figs. 868 and 869 are vertical sections through the head and sill, and Fig. 870 is a horizontal section through the frame for the double-hung sashes A two-light window opening in an 18-in. brick wall having a stone mullion and stone dressings is shown in horizontal section by Fig. 871. Enlarged details of one of the cased frames, with double-hung sashes, are illustrated in Fig. 872. Figs. 873 and 874 show in sectional plan and in elevation one jamb of a window, 4 ft. wide, in an 18-in. brick wall, the sash frame being set behind a half-brick reveal. Folding shutters and boxings, as in the best work, are shown, and portions of a section through skirting, window back, and sash, with soffit, are also given. Portions of elbows and shutters are given in elevation by Fig. 874. Windows are usually set back from the face of the wall by a 4½-in. reveal.

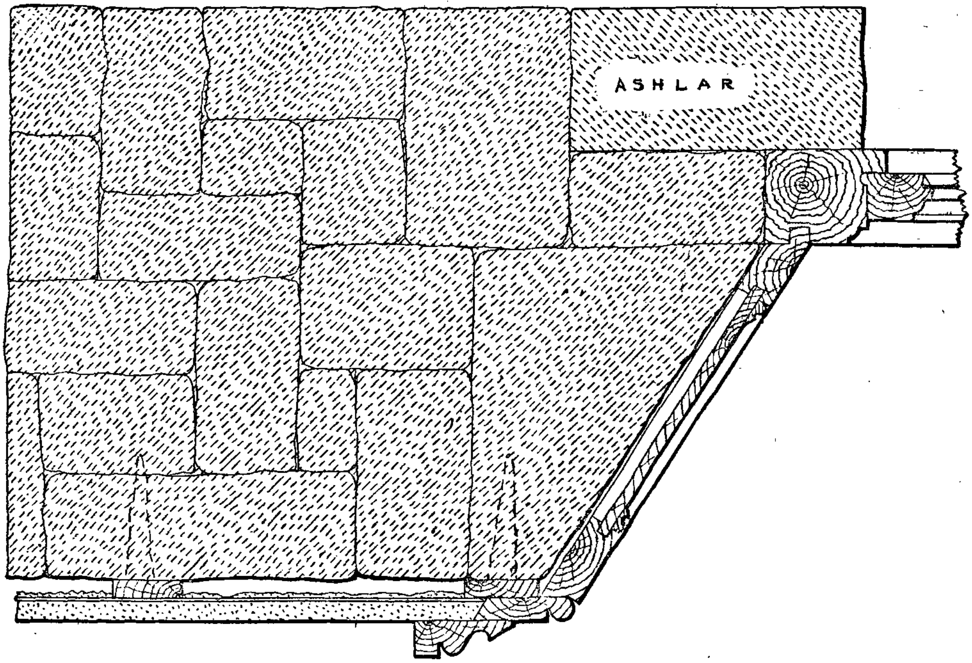
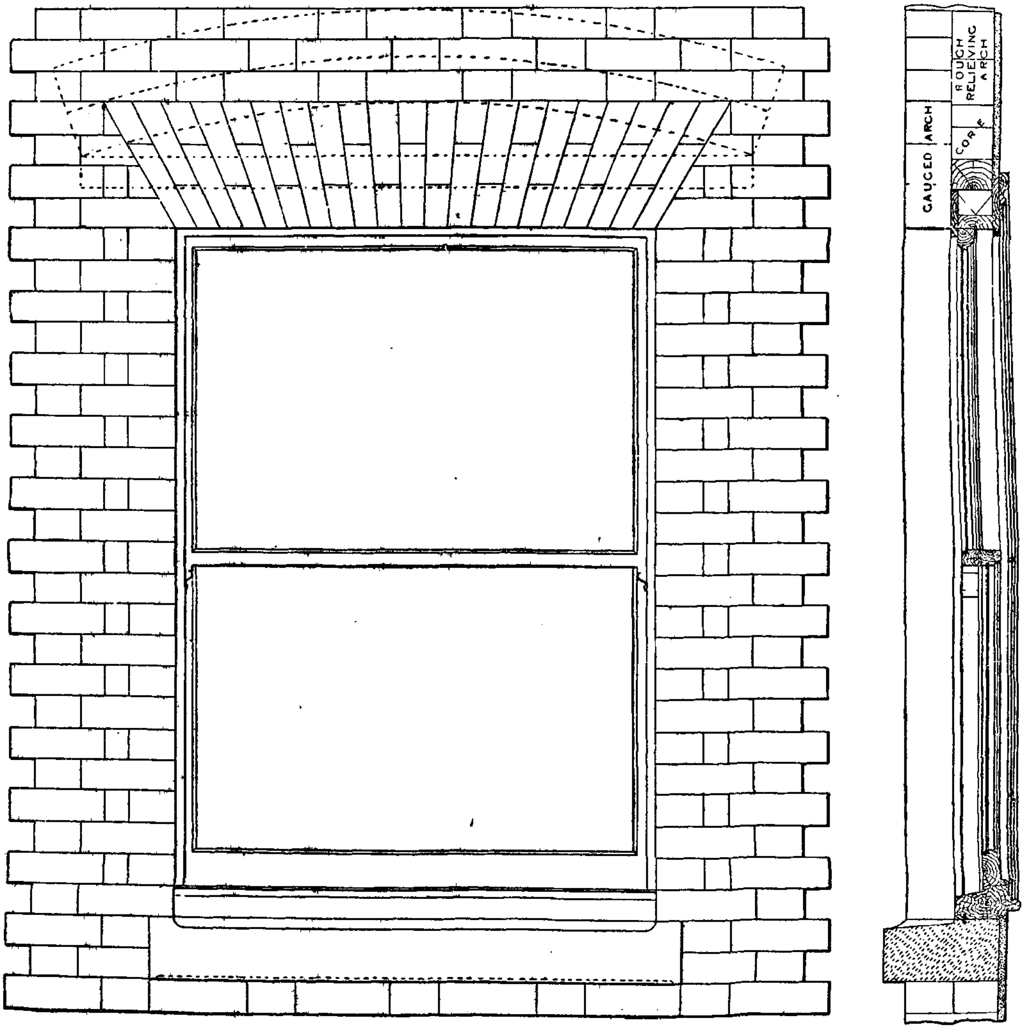


Fig. 878.—Horizontal Section through One Side of French Casement Window Opening.

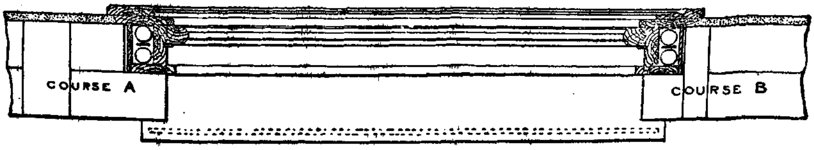
Window Openings in Stone Walls.

The horizontal section of a fully trimmed window, through one jamb and reveal, is presented by Fig. 875. The wall is of stone, ashlar face, reveal 6 in. deep, wall 2 ft. thick, battened (or stoothed); a jamb lining, three-faced architrave folding shutters (window about 4 ft. wide), ground, lathing, and plaster, are also shown in Fig. 875. A vertical section through sash, bottom rail, oak sill, and stone sill is presented by Fig. 876. It is not usual to make any special provision to prevent the water from passing the end of a water bar, as very little can reach that point. The oak sill itself may be bedded on whitelead as well as the water bar, or the stone sill may be rebated as well as weathered, the oak sill overhanging the edge of the rebate and throated to prevent the passage of moisture. A horizontal section through a 5-ft. two-light window opening in a 24-in. stone wall is presented by Fig. 877, this showing a stone mullion, and in one light the details of a



ELEVATION OF DOUBLE HUNG SASH

VERTICAL SECTION



HORIZONTAL SECTION

Figs. 879 to 881.—Elevation and Sections of Double-hung Sash.

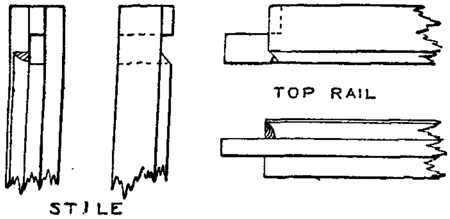


Fig. 882.—Joint of Top Rail and Stile.

cased frame and double-hung sash, together with splayed jamb linings with moulded panels. Fig. 878 shows a horizontal section through one side of a French casement 4-ft. window opening, in a wall built of coursed rubble with ashlar dressings. The frame is solid, and the hanging stile of the sash opens inwards; the jamb lining has moulded panels and double-faced architraves. Lath and plaster on battens plugged to the wall is also shown.

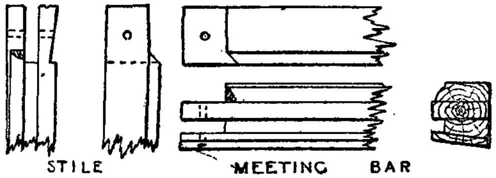


Fig. 883.—Joint of Meeting Bar and Stile.

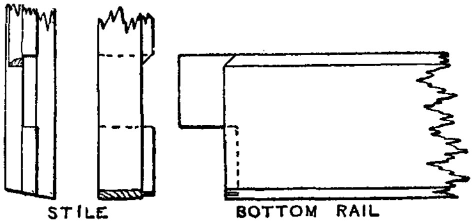


Fig. 884.—Joint of Bottom Rail and Stile.

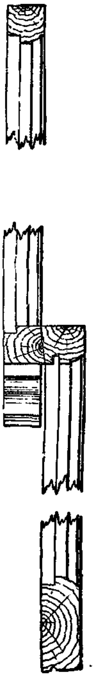
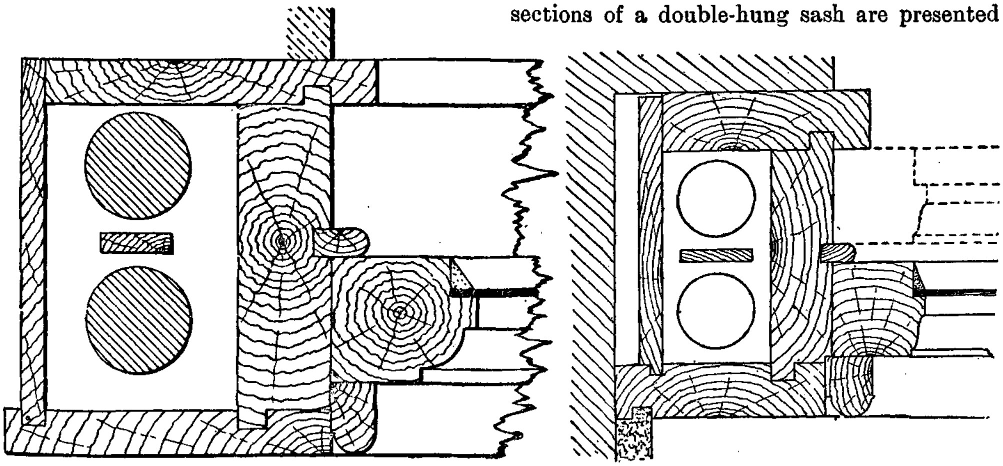


Fig. 885.—Enlarged Section through Window Sash.

Window Sashes.

An elevation and vertical and horizontal sections of a double-hung sash are presented



Figs. 886 and 887.—Horizontal Sections through Cased Frames, Pulley Stiles, etc.

by Figs. 879 to 881; enlarged details of the joints, etc., in the sashes being given by Figs. 882 to 885. Sections on a larger scale through very similar pulley stiles, etc., are given by Figs. 886 and 887; the former (Fig. 886) shows how the glass is fixed, and the following details :—⅝-in. inside and outside linings; 1¼-in. pulley piece; ⅜-in. back lining; ⅜-in. parting bead; 1-in. by ⅝-in. inside bead; ¼-in. parting slip. The inside bead or beads (sometimes called the guard beads) should be fixed with brass screws, or cups and screws, to allow of being readily removed when required. The rebate (pronounced "rabbet" by workmen) is for fixing the width of opening to suit thickness of the sash, which might otherwise be carelessly done, leaving the sash too tight requiring it to be eased, or too loose allowing it to rattle, or tight one end and loose the other. A Yorkshire light is shown in horizontal and vertical sections by Figs. 888 and 889. A vertical section of a sash hung on centres is presented by Fig 890. The proper position of the cut in the beads should be noted, part of each being fixed to the sash.

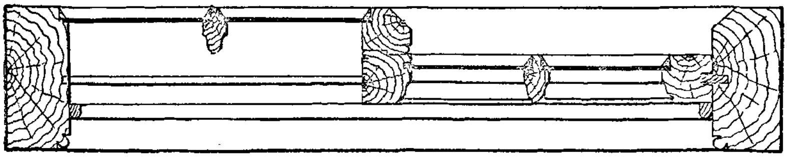


Fig. 888.—Sectional Plan of Yorkshire Light.

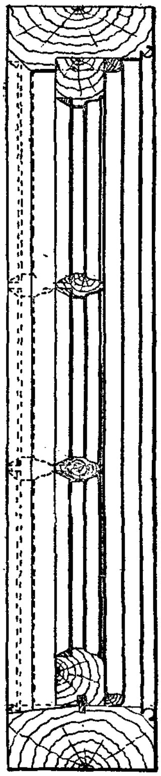


Fig. 889.—Vertical Section of Yorkshire Light.

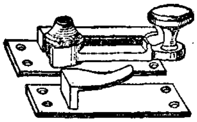


Fig. 891.—Brass Sash Fastener for Double-hung Sashes.

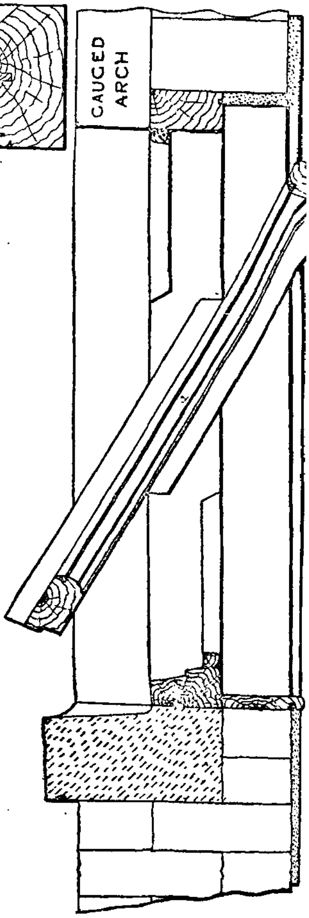


Fig. 890.—Section of Sash hung on Centres.

Sash Fastener.

In order to prevent double-hung sashes from rattling, the inside bead and bottom rail of sash are bevelled, the meeting bars bevelled and rebated, and the brass sash fastener (Fig. 891) is fixed; this is strongly made and so shaped as to lift the top sash and push down the bottom one while binding them firmly together. A further point of importance in preventing rattling is that the work should be true and the sashes should only have just sufficient clearance to slide freely.

Casement Windows.

An elevation and a plan of a French casement with fanlight are given respectively by Figs. 892 and 893, alternative shapes for the sash bars being shown by Figs. 894 to 899. It will be noted that a cross section through

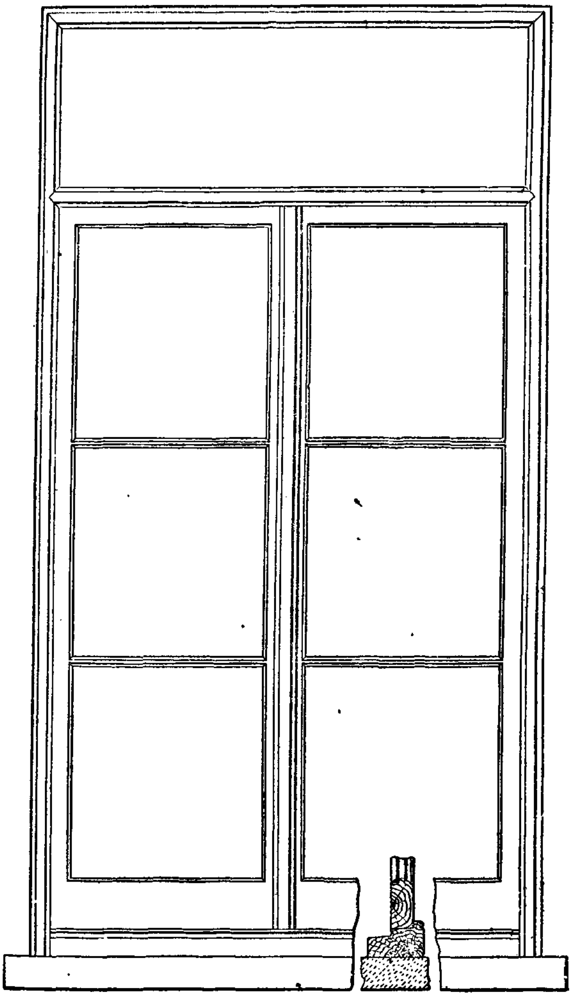


Fig. 892.—Elevation of French Casement with Fanlight.

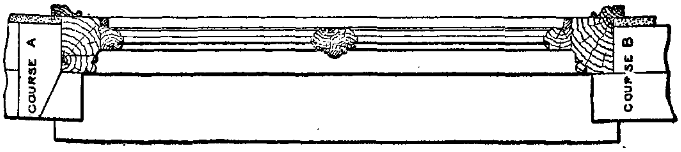


Fig. 893.—Plan of Casement.

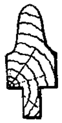


Fig. 894.—Lamb's Tongue Sash Bar.

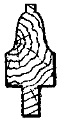


Fig. 895.—Lamb's Tongue and Fillet Sash Bar.

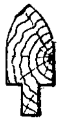


Fig. 896.—Gothic Sash Bar.

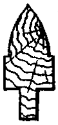


Fig. 897 --Gothic and Fillet Sash Bar.

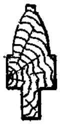


Fig. 898.—Ovolo and Fillet Sash Bar

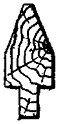


Fig. 899.—Rustic or Bevelled Sash Bar.

the bottom rail and sills is given at the bottom of Fig. 892. Another elevation of a casement window, 4 ft. by 2 ft. 6 in. (sash size), is presented by Fig. 900. This is a single sash in four panes. The hinges and fasteners to be used are made of wrought iron in common work, and of brass in best work. They consist of the cockspur fastener (Fig. 901), the casement stay and nipple (Fig. 902), and a pair of butt hinges (Fig. 903). A French casement is generally arranged like folding doors, one side fixed by bolts top and bottom and the other by a mortice lock, a casement fastener, or espagnolette bolt (Fig. 904). It is important that a casement should be tightly closed, as otherwise draught and moisture will pass through. Hook rebates are used in the joints to minimise this defect.

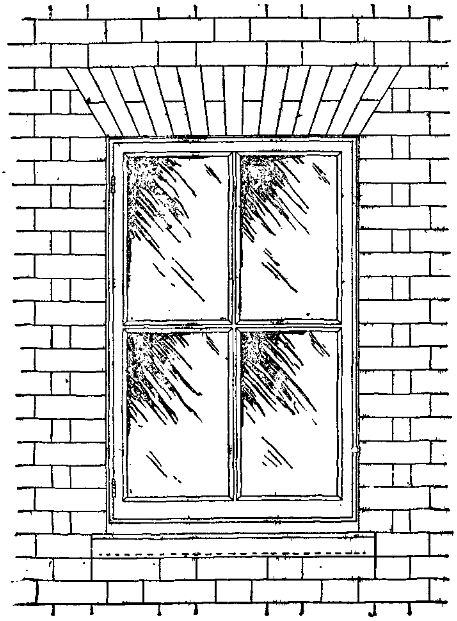


Fig. 900.—Elevation of Casement Window.

Junction of Soffit and Jamb Lining.

The common method of connecting rebated and beaded jamb linings to the square soffit lining is shown in Fig. 905, while Fig. 906 shows a method when the rebate and bead are both worked on the solid. In the former a part of the architrave is shown in position; in the latter it is removed.

Specification of Square-headed Sash Window in Dining-Room.

The following is a specification of a square-headed sash window in an 18-in. wall of a handsomely fitted dining-room; with stone sill, boxing shutters, and the usual joinery complete (no sash bars). Glazing, ironmongery, and painter's work to be described; the woodwork to be grained in oak, the panels where most seen being more richly grained than elsewhere. The windows are assumed to be 4 ft. by 6 ft. *Bricklayer.*—Cut and pin Portland stone sills to dining-room windows, and make good to facings. Build in coke-breeze fixing blocks 2 ft. apart to all window openings, for attachment of joinery. *Mason.*—Put to dining-room windows 9-in. by $3\frac{1}{2}$-in. Portland stone, rubbed, sunk, weathered, and throated sills, grooved for iron water bar, and with proper stools for jambs. *Joiner.*—Provide and fix to dining-room window openings superior deal cased frames with 3-in. sunk, weathered, throated, and twice grooved oak sills; $1\frac{1}{4}$-in. grooved pulley stiles and head, all tongued to 1-in. grooved inside and outside linings, $\frac{3}{8}$-in. oak parting beads, $\frac{5}{8}$-in. by 1-in. staff beads (with 3-in. beaded ventilating piece to sills); $\frac{1}{2}$-in. back linings, $\frac{1}{4}$-in. parting slips; each frame to be provided with 2-in. brass-faced axle pulleys, and fitted with 2-in. rebated and moulded sashes, double hung with best flax

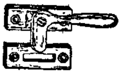


Fig. 901.—Casement Cockspur Fastener.

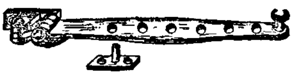


Fig. 902.—Casement Stay and Nipple.

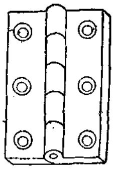


Fig. 903.—Butt Hinge.

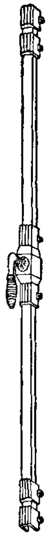
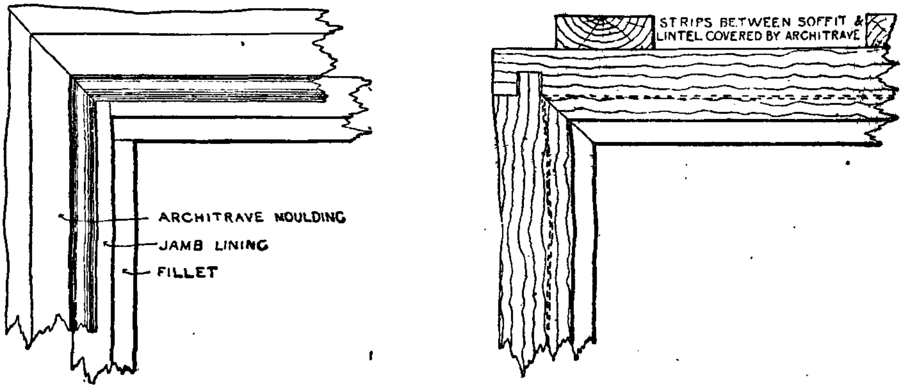


Fig. 904.—Espagnolette Bolt.

lines and iron weights ; the meeting rails to be 1¾ in. thick, bevelled and rebated, and the stiles of top sashes to have moulded horns ; the whole to be put together in the best manner according to the details to be provided. Provide and fix 1¼-in. tongued and rounded window boards to window frames, grooved, and both ends splayed for window backs. Provide and fix 1¼-in. framed and moulded window backs in one panel, canvas-backed and painted in red-lead, with framed and moulded elbows on splay, tongued to window board and elbow caps. Provide and fix 1¼-in. moulded and bead flush boxing shutters, hung folding in two heights, with 1-in. bead flush and square back flaps, including all rebates and splay rebates, in proper shutter boxings framed on splay, 1-in. framed and moulded back lining two panels high and twice tongued. Provide and fix 1-in. elbow caps, both ends splayed and front edge rounded ; and ¾-in. soffits to boxings, both ends splayed. Provide and fix to windows 6-in. double-faced architrave moulding, mitred at angles and ends housed. Provide and fix all further linings, moulded as


ARCHITRAVE MOULDING
JAMB LINING
FILLET
STRIPS BETWEEN SOFFIT & LINTEL COVERED BY ARCHITRAVE

Figs. 905 and 906.—Connecting Jamb Linings to Soffit Lining.

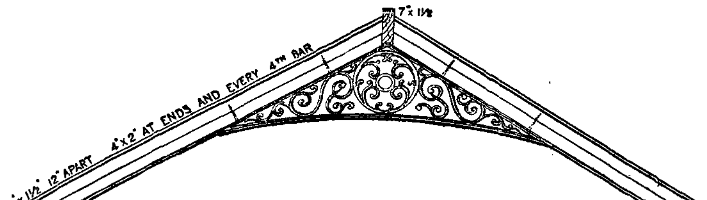

7"x 1½"
4"x2" AT ENDS AND EVERY 4TH BAR
3½"x 1½" 12" APART
SILL AT END

Fig. 907.—Span Roof or Couple Roof, suitable for Greenhouses.

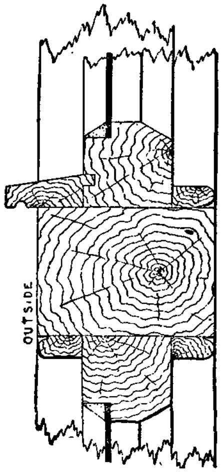


Fig. 908.—Section through Sill at End.

required to correspond with adjoining work to form a proper finishing. *Ironmonger.*—Provide and fix to dining-room sashes 3-in. Hopkinson's patent strong brass sash fastenings, to bottom rail of lower sash a pair of strong brass sash-lifts, and to top rail of upper sash a sunk brass eye for long-arm. Hang boxing shutters and back flaps with 2-in. cast-iron butts, and fix black china screw knobs to front shutters. *Glazier.*—Glaze the dining-room windows; lower sash with ¼-in. British polished plate glass, best glazing quality, well bedded and back puttied and edges blacked, upper sash with 21-oz. plate glass, second quality. *Painter.*—*Inside.*—Knot, prime with lead and oil priming, well stop, and paint four oils, well rubbed down between each coat on all exposed woodwork; grain oak, and twice varnish with best copal varnish. The panels of shutters, window back, and elbows to be grained and over-grained to imitate pollard oak. All concealed woodwork to have two coats of oil colour before being fixed. *Outside.*—Knot, prime, stop, and paint four oils on woodwork, and finish approved tint.

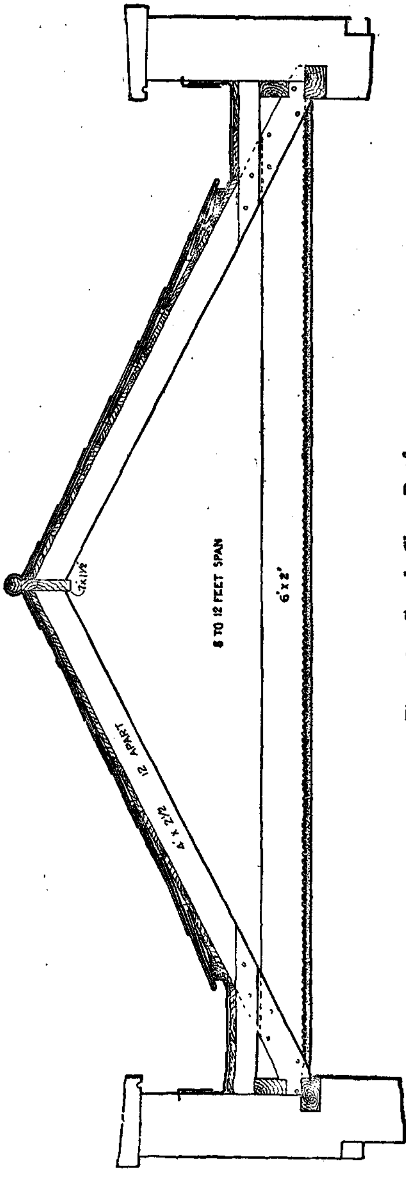


Fig. 909.—Couple-Close Roof.

Small Roofs.

Small roofs for conservatories and other lightly constructed buildings may be very simple. The span roof or couple roof (Fig. 907) is available for spans not exceeding 12 ft., and is particularly useful for greenhouses. A section through the sill at the end is given by Fig. 908. The couple-close roof (Fig. 909) may be compared with Fig. 907; it is suitable for any span ranging from 8 ft. to 12 ft. The simplest form of collar-beam roof is that shown by Fig. 910; this is suitable for a span of from 8 ft. to 18 ft. Alternate methods of jointing the collar to the rafter are shown by Figs. 911 and 912. The couple-close roof with collar and suspenders (Fig. 913) is suitable for a span of from 14 ft. to 18 ft. The lean-to roof (Fig. 914) is the simplest of all, but its use is limited to sheds, etc., and its span should not exceed 8 ft.; such a roof is covered with asphalted roofing felt obtainable in 35-ft. lengths, 32 in. wide and ⅛ in. thick. It is laid with a 2-in. lap, and is nailed with 1-in. strong clouts or with felt tacks. After laying, the felt should be coated with coal tar boiled with slaked lime, powdered chalk, or whiting (proportions, three and one), and then well sanded. A lean-to roof can be strengthened by supporting it with a rolled joist, and thus may answer for a span of 19 ft. 6 in. Assume a slated lean-to roof with 9-in. by 3-in. rafters, 18 in. apart, the roof having a clear span of 19 ft. 6 in. The bearings must be horizontal, as shown in Fig. 915. Taking the total load of the structure, wind and snow at ½ cwt. per ft. super., the load on each rafter

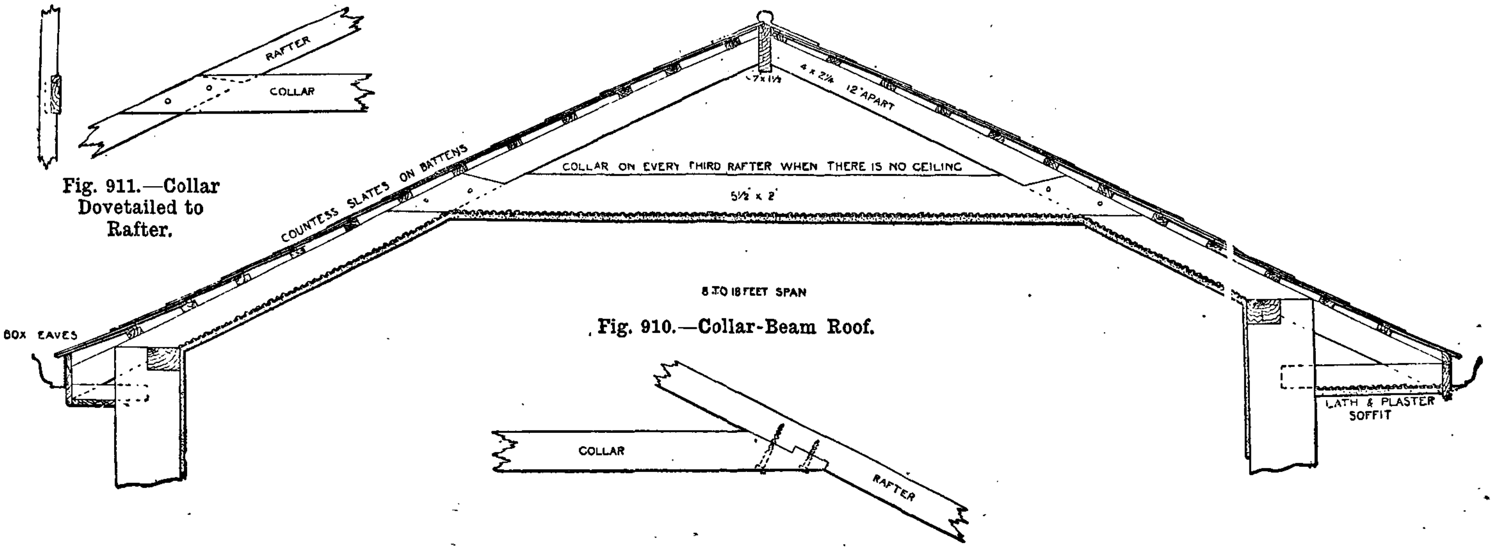

RAFTER
COLLAR

Fig. 911.—Collar Dovetailed to Rafter.

7 x 1½
4 x 2¼ 12' APART
COLLAR ON EVERY THIRD RAFTER WHEN THERE IS NO CEILING
5½ x 2
COUNTESS SLATES ON BATTENS
BOX EAVES
LATH & PLASTER SOFFIT
8 TO 18 FEET SPAN

Fig. 910.—Collar-Beam Roof.

COLLAR
RAFTER

Fig. 912.—Collar Notched to Rafter.

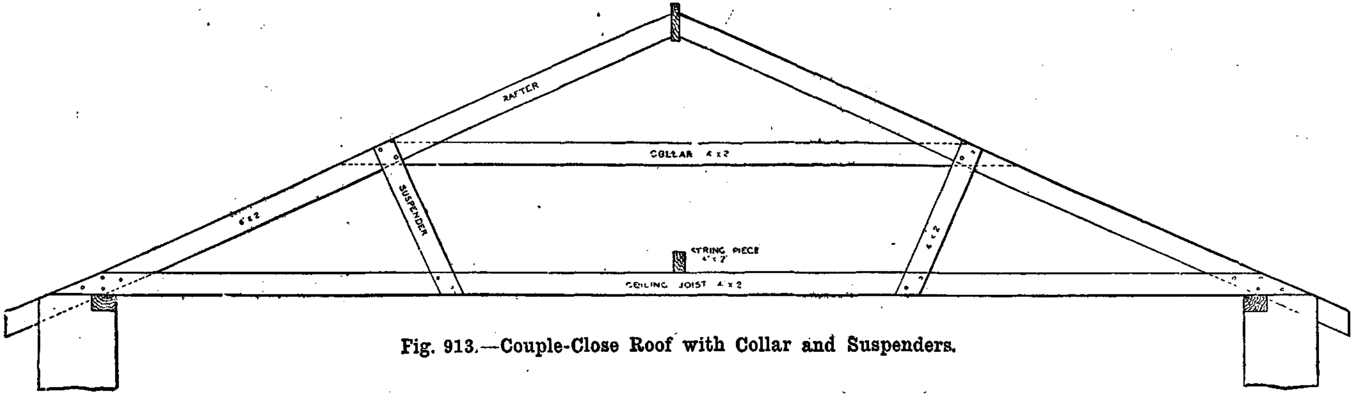

RAFTER
COLLAR 4 x 2
SUSPENDER
4 x 2
4 x 2
STRING PIECE 4 x 2
CEILING JOIST 4 x 2

Fig. 913.—Couple-Close Roof with Collar and Suspenders.

at 19-ft. 6-in. span and 18-in. centres would be $19\cdot5 \times 1\cdot5 \times \frac{1}{2} =$ say 15 cwt. The breaking weight on a beam, cwt. in centre, $= \frac{c\,b\,d^2}{L}$ for fir $c = 3\cdot5$, therefore, with 9-in. by 3-in. rafters $\frac{3\cdot5\,b\,d^2}{L} = \frac{3\cdot5 \times 3 \times 9^2}{19\cdot5} =$ say 44 cwt. Distributed load twice central load say $44 \times 2 = 88$ cwt. Factor of safety say 5 to 1, then safe load $= \frac{88}{5} = 17\cdot6$ cwt., as against 15 to be provided for, which is just sufficient margin. The rolled joist must be strong enough to carry a working load of 12 cwt. per foot span, and rest on a stone template. The piers must be suitable for the load upon them. Their size will depend largely on the length of the girder.

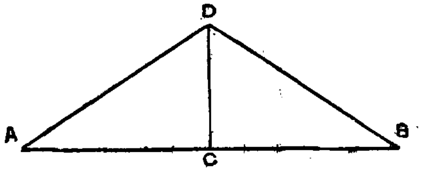


Fig. 916.—Setting Off Pitch of Roof.

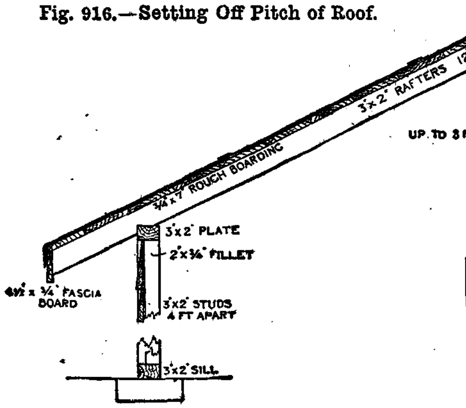


Fig. 914.—Lean-to Roof for Sheds, etc.

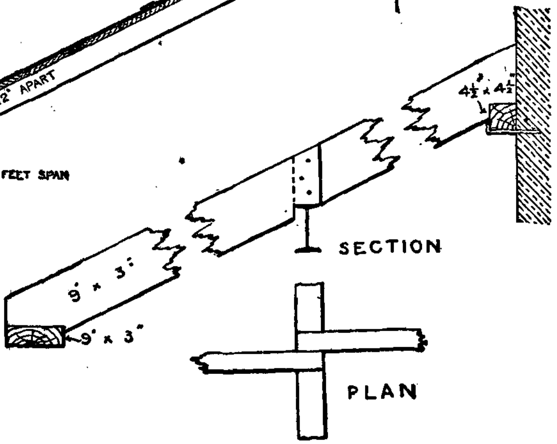


Fig. 915.—Lean-to Roof Supported by Girder.

Collar-Beam Trusses.

A collar-beam truss has already been shown (Fig. 910, p. 227). Collar braces or collar beams are timbers placed horizontally from one-third to half-way up a roof between the rafters on each side, and connected by halving, tenoning, or merely nailing. They are usually of the same scantling as the rafters. The principal rafters in a collar-beam truss are usually birdsmouthed on to a continuous fir wall plate, 4½ in. by 3 in., either on the inner edge or centre of the wall, or notched over it and continued when on the outer edge; but in the case of an exceptionally heavy roof a 6-in. by 4-in. wall-plate would be more suitable. Stone templates might possibly be adopted, say 18 in. by 9 in. by 4 in., instead of the wall-plates, but the writer has never seen them used for a collar-beam truss. All collar-beam roofs exert a thrust upon the walls; and if these yield, a great strain is thrown on the truss at the junction of the collar and rafters. If a truss be laid down upon a floor with the feet touching one wall and the head about 18 in. or 2 ft from the opposite wall, a screw-jack may be inserted between the wall and the head, when the spreading of the feet would soon break the rafters where the collar joins, and show the effect of the load in causing a thrust upon the walls after erection. With a tile roof at 45°, and trusses 18-ft. span placed every 10 ft., the dead load upon one truss would be about 2½ tons, and the effect of the wind about the same amount additional.

Pitch of Roof.

Amount of Pitch.—The pitch of a roof depends upon the ideas of the architect. For

Gothic work and for exposed positions, high-pitched roofs are used, say 45° or 60°, and occasionally more, covered with shingles, slates, or plain tiles. For ordinary buildings, either 30° or 26½° is adopted for the pitch, the former having a rise of half the length of rafter, and the latter having a rise of one-fourth the span, known also as square pitch. Sometimes, for large sheds with iron roof trusses, the pitch is reduced still more—to (say) a minimum of 22½°. The flatter the roof, the heavier the slates should be, and the lap should also be increased. Where a 2½-in. lap would do for a pitch of 60°, a 4-in. lap would be desirable for a pitch of 22½°.

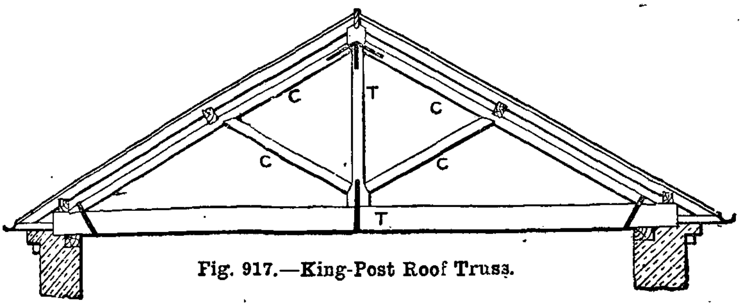


Fig. 917.—King-Post Roof Truss.

Determining Pitch.—When the span and rise are given, the pitch (*a*) will be $\frac{\text{rise}}{\text{span}}$ (for example, 24-ft. span 6-ft. rise $=\frac{6}{24}=\frac{1}{4}$ pitch); or (*b*) will be a slope of $\frac{\frac{1}{2}\text{ span}}{\text{rise}}$ to 1 (for example, in the given case $\frac{\frac{1}{2}\times 24}{6}=2$ to 1); or (*c*) the pitch in degrees will be the angle whose tangent is $\frac{\text{rise}}{\frac{1}{2}\text{ span}}$ (for example, in the given case $\frac{6}{\frac{1}{2}\times 24}=\cdot 5$), which is the tangent of an angle of 26° 33′.

Setting Off Pitch.—To set off the slope of a roof at one-third pitch draw the span A B (Fig. 916), divide it into three equal parts, and at the centre of the span C set up the perpendicular C D equal to one part join A D. Then A D will be the required pitch. This is the flattest pitch at which tiles should be laid.

Applications of King-Post and Queen-Post Trusses.

A king-post truss (Fig. 917) is used up to 30-ft. span and a queen-post truss (Fig. 918) beyond, so that the unsupported length of principal rafter shall not exceed the safe limit, also that the purlins may not be too far apart for common rafters of usual section. For purposes of comparison, Figs. 917 and 918 are given. Members subject to compression are indicated by C, and to tension by T. The sizes, or scantlings, for the members of these trusses are usually taken from tables.

Details of King-Post Roof Truss.

Fig. 919 is a cross section through a little over one-half of a 24-ft. span roof, resting on 14-in. brick walls, showing the roof truss ceiled to the underside of the tie-beam. The eaves overhang, and are finished with soffit and fascia boarding, and ogee cast-iron gutter. Slates are shown, and all scantlings are dimensioned. Certain details of construction require to be shown on a larger scale. The method of securing the foot of the king-post to the tie-beam by means of a stirrup is shown by Figs. 920 and 921. A bolt (Figs. 922 and 923) is less common. The gibs and collars, together with the proper clearances, are shown at the top of the stirrup (Figs. 920 and 921). The king-post, rafters, and ridge are jointed as shown in Fig. 924, and two forms of iron strap for securing the king-post to the two principal rafters are illustrated in Figs. 925 and 926. In Fig. 919 the principal rafter is bolted to the tie-beam, but two other ways are

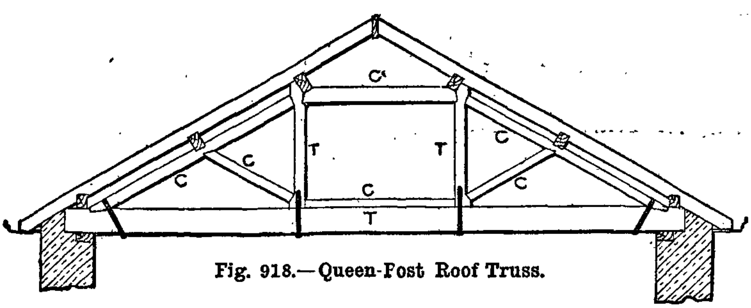
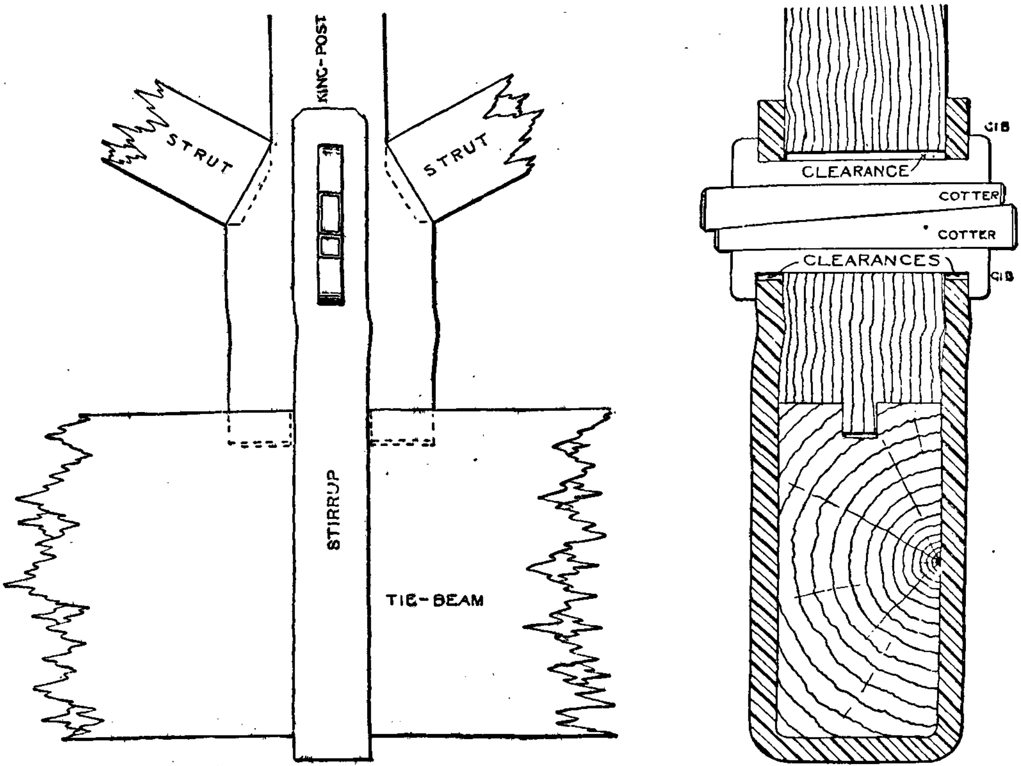
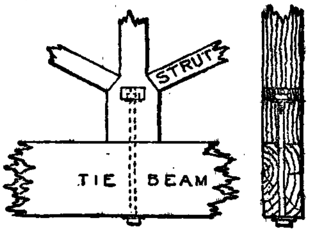


Fig. 918.—Queen-Post Roof Truss.


KING-POST
STRUT
STRUT
STIRRUP
TIE-BEAM
GIB
CLEARANCE
COTTER
COTTER
CLEARANCES
GIB

Figs. 920 and 921.—Foot of King-Post Secured to Tie-Beam with Iron Stirrup.


STRUT
TIE BEAM

Figs. 922 and 923.—Foot of King-Post Bolted to Tie-Beam.

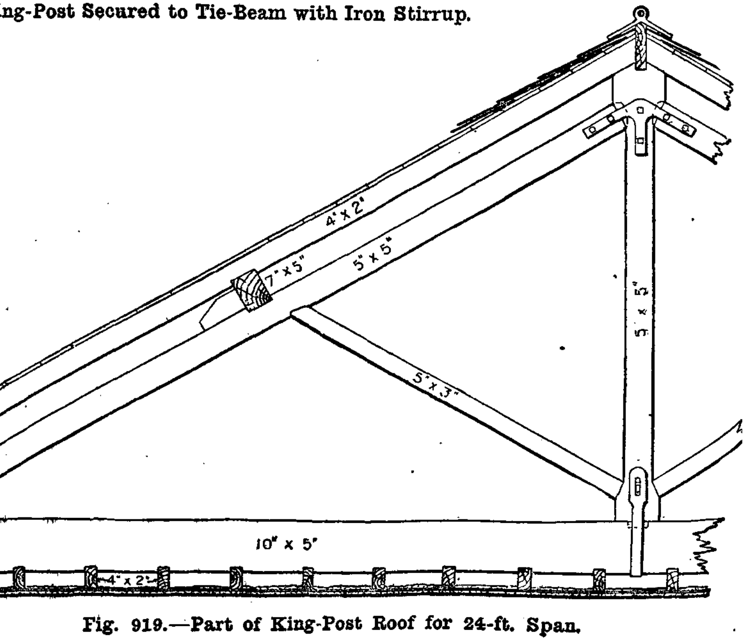

4"x2"
7"x5"
5"x5"
5"x5"
5"x3"
6"x4"
10"x5"
4"x2"

Fig. 919.—Part of King-Post Roof for 24-ft. Span.

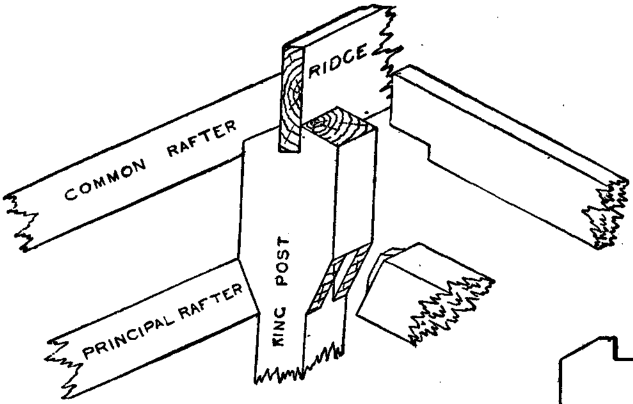

RIDGE
COMMON RAFTER
KING POST
PRINCIPAL RAFTER

Fig. 924.—Joint of King-Post, Rafters, and Ridge.

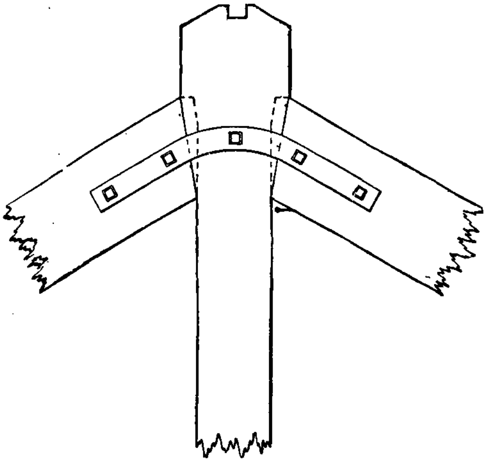


Fig. 925.—Joint of King-Post and Rafters Strengthened with Strap.

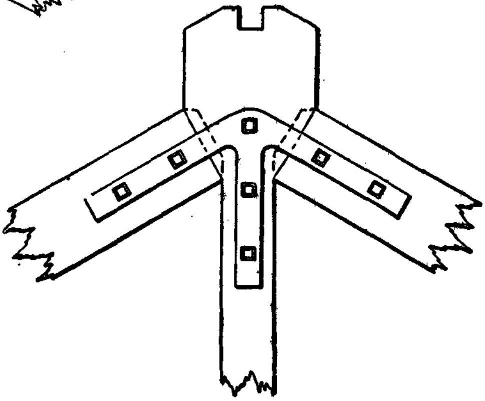


Fig. 926.—Joint of King-Post and Rafters Strengthened with Strap.

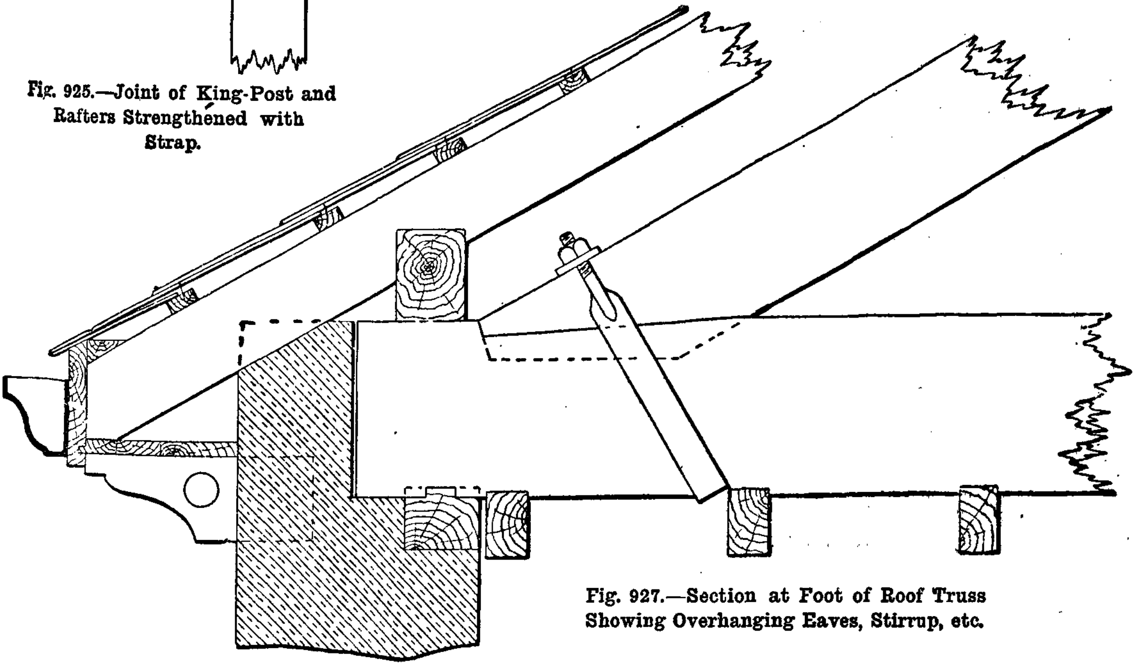


Fig. 927.—Section at Foot of Roof Truss Showing Overhanging Eaves, Stirrup, etc.

possible. It may have a stirrup as in Fig. 927, or a heel strap as in Fig. 928. The bolt is better than the heel strap, but the stirrup is best. This is because the strap cannot be properly tightened; while the bolt may be tightened, but it weakens the timbers a trifle by loss of sectional area, which, however, is not at all serious. The stirrup weakens the timber least, and can be tightened up readily. For shapes of heel straps and stirrups, see Figs. 929 and 930. The method of jointing the rafters to the pole plate and tie-beam and the latter to the wall plate is clearly shown in Fig. 931. The cleat used in fixing the purlin to the principal rafter is shown in Fig. 932.

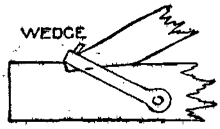


Fig. 928.—Heel Strap at Foot of Roof Truss.

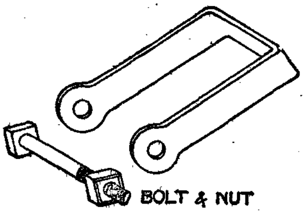


Fig. 929.—Heel Strap.

Queen-Post Truss.

Fig. 933 is a section through about one-half of an ordinary queen-post roof over a dwelling-house with 18-in. stone walls, the span being 34 ft. The figure shows a stone blocking course and cornice with gutter and lead ridge, also four courses of duchess slates on boards, both at eaves and ridge. Many of the details of this truss are exactly the same as those of the king-post truss already illustrated so fully. The principal differences are that in this case a horizontal straining beam has to be jointed to the queen-posts; the joint is clearly shown by Fig. 934. Figs. 935 and 936 show other forms of this joint, and there is not a very great difference in their respective strengths; but Fig. 936 represents the more usual and the better design, because the main stresses are bounded by the tie-beam, the principal rafters up to the straining beam, and the straining beam itself. The portion of truss above the straining beam has only a very small stress upon it, and it is therefore unnecessary to make the upper ends of principal rafters of such large scantling as the lower ends, which form an essential part of the truss. The stirrup at the foot of the queen-post is shown by Figs. 937 and 938. At the foot of the principal rafter is the bridle joint illustrated by Fig. 939. A queen-post truss suitable for a roof of 37-ft. span to be covered with Welsh slates is shown by Fig. 940. A queen-post roof strengthened with struts and iron rods is shown by Fig. 941; this is suitable for a roof of 50-ft. span on a public building. This form of truss can be adopted for an open timber roof if the lower part is

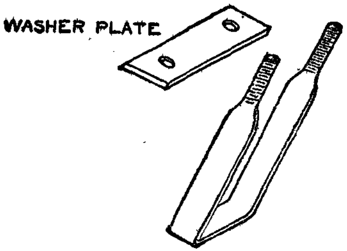


Fig. 930.—Stirrup.

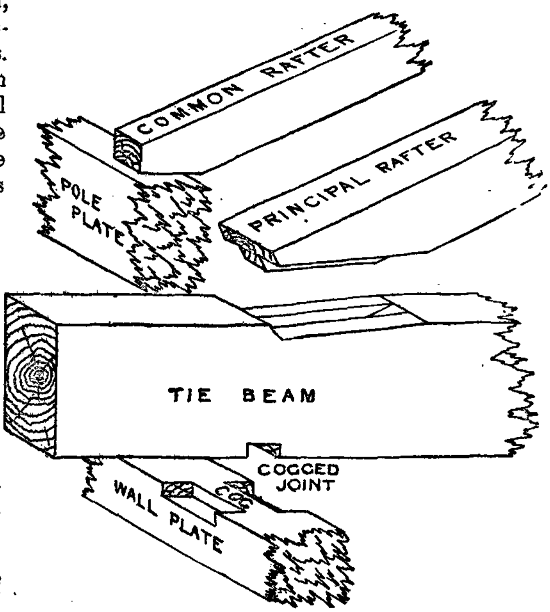


Fig. 931.—Joints at Foot of Principal Rafter.

removed. In Fig. 941 the construction of the ceiling is assumed to be entirely concealed by a coved plaster ceiling.

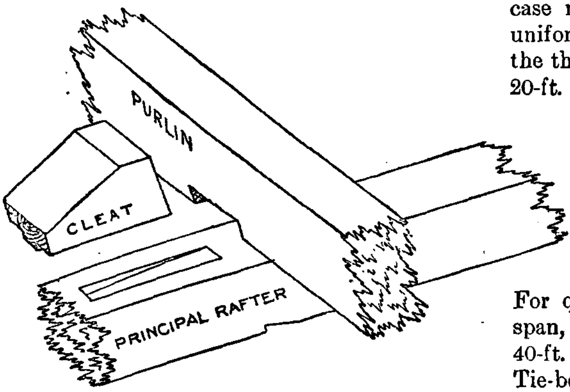


Fig. 932.—Securing Purlin to Rafter with Cleat.

King- and Queen-Posts used without Side Cutting.—The method of using the king- and queen-posts without cutting at the sides is common throughout Yorkshire, but is never seen in London except in the roughest work. It adds to the weight of the truss, does not increase the strength, and is considered unsightly, but saves a little labour. The purlins should be at right angles to the principal rafter, to take the load from the common rafters, which, being held at the top, press in that direction, and not vertically.

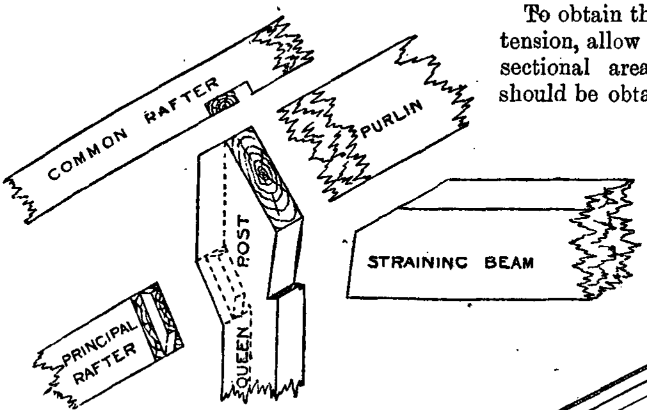


Fig. 934.—Joints at Head of Queen-Post.

Determining Scantlings of Roof Trusses.

There is no common rule for scantlings of roof trusses, but the writer suggests that the case may be simplified by making trusses of uniform thickness throughout all the members, the thickness being 4 in. for a king-post truss, 20-ft. span, advancing ½ in. for each 2 ft. of additional span up to 30 ft. Tie-beam twice thickness in depth, struts half thickness in depth, king-post shank same as struts + 1 in., head twice the width of the shank, principal rafters 4 in. deep for 20-ft. span, adding ⅛ in. for each 2 ft. beyond. For queen-post trusses, 4 in. thick for 32-ft. span, increased by ½ in. for every 2-ft. up to 40-ft. span, and by ¼ in. for every 2 ft. beyond Tie-beam twice thickness + 1 in. in depth, queen-post shank square, struts half thickness in depth, principal rafters square, straining beam 3 in. less depth than tie-beam, straining sill 3 in. thick.

Calculating Wrought-Iron Roof Trusses.

To obtain the sectional area of members in tension, allow 1 sq. in. to 5 tons stress. The sectional area of members in compression should be obtained by Gordon's formula:

$$w = \frac{16\,s}{1 + \dfrac{l^2}{a\,d^2}}$$

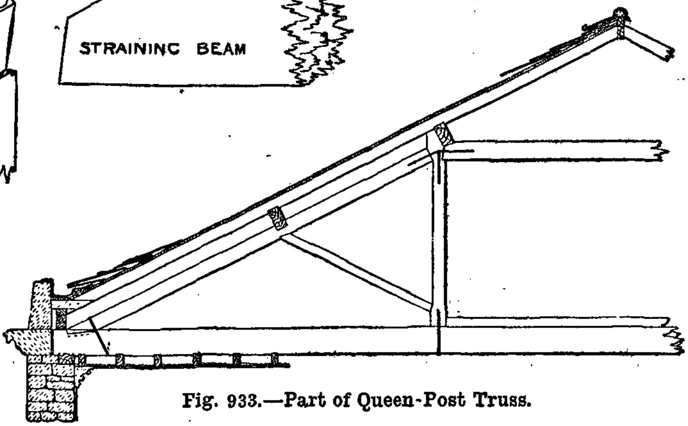
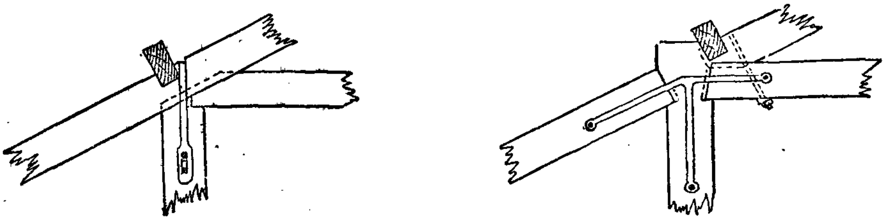


Fig. 933.—Part of Queen-Post Truss.

w = breaking weight on section; s = sectional area, square inches; l = length, inches; d = least diameter or breadth, inches; a = constant = 1,500 for cross angle or tee fixed both ends:

= 750 for ditto fixed one end, jointed the other; = 375 for ditto jointed both ends. Factor of safety, ¼.



Figs. 935 and 936.—Joint of Straining Beam, Principal Rafter, and Queen-Post.

Calculating Timber Roof Trusses.

The sizes of timbers for a roof truss can be calculated or found from a stress diagram; but the more usual plan is to refer to tables such as those given below. The common rule is to allow for wind and weight of covering ½ cwt. per ft. super. acting vertically on the surface of the roof. Open timber roofs, and those in exposed positions, require to be specially calculated.

Jointing Wall Plates.

The best manner of jointing wall plates at the angle of a building is probably plain double notching with 4 in. left solid beyond notch, as in Fig. 942. This forms a good tie for the walls, and when no projecting ends can be left the parts may be cut as in Fig. 943.

SCANTLING FOR KING-POST TRUSSES.

Baltic fir, 10 ft. apart. Pitch, 25° to 30°. Slated.

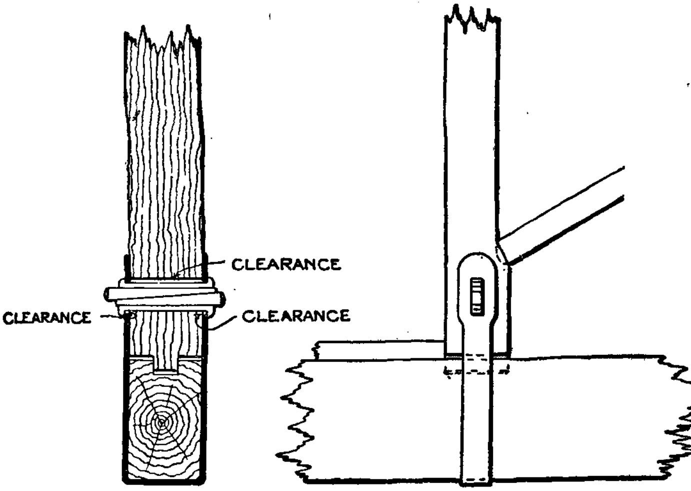
Span.	Thickness of Truss and all Members.	Breadth on Elevation.				Purlins.	Common Rafters.
		Tie-Beam.	King-Post.	Struts.	Principal Rafters.		
Ft.	In.	In.	In.	In.	In.	In.	In.
20	4	8	3	2	4	8 × 4	3½ × 2
22	4½	9	3½	2¼	4	8 × 4	4 × 2
24	5	10	3½	2½	4	8 × 4½	4 × 2
26	5½	11	4	2¾	4	9 × 4½	4½ × 2
28	6	12	4	3	4	9 × 5	4½ × 2
30	6½	13	4½	3	4½	9 × 6	5 × 2

Head and foot of king-post, twice width of middle. Reduce thickness of truss by ½ in. and depth of tie-beam by 1 in. if there is no ceiling.

SCANTLING FOR QUEEN-POST TRUSSES.

Baltic fir, 10 ft. apart. Pitch, 25° to 30°. Slated.

Span.	Thickness of Truss and all Members.	Breadth on Elevation.					Purlins.	Common Rafters.
		Tie-Beam.	Queen-Post.	Struts.	Principal Rafters.	Straining Beam.		
Ft.	In.	In.	In.	In.	In.	In.	In.	In.
32	4	9	4	2	4½	6	8 × 4	3½ × 2
34	4½	10	4	2	5	6½	8 × 4	4 × 2
36	5	10½	4½	2	5½	7	8 × 4½	4 × 2
38	5½	11	4½	2	5½	7½	8 × 4½	4 × 2
40	5½	11½	5	2½	6	8	9 × 4½	4½ × 2
42	6	12	5	2½	6	8	9 × 4½	4½ × 2
44	6	12½	5½	2½	6½	8½	8 × 5	4½ × 2
46	6	13	6	2½	7	9	9 × 6	5 × 2


CLEARANCE
CLEARANCE
CLEARANCE

Figs. 937 and 938.—Foot of Queen-Post Secured to Tie-Beam with Stirrup and Gibs and Cottars.

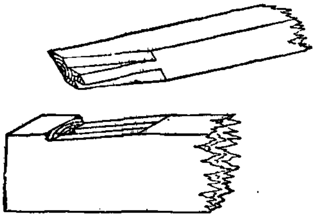


Fig. 939.—Tenon Joint at Foot of Rafter in Queen-Post Roof.

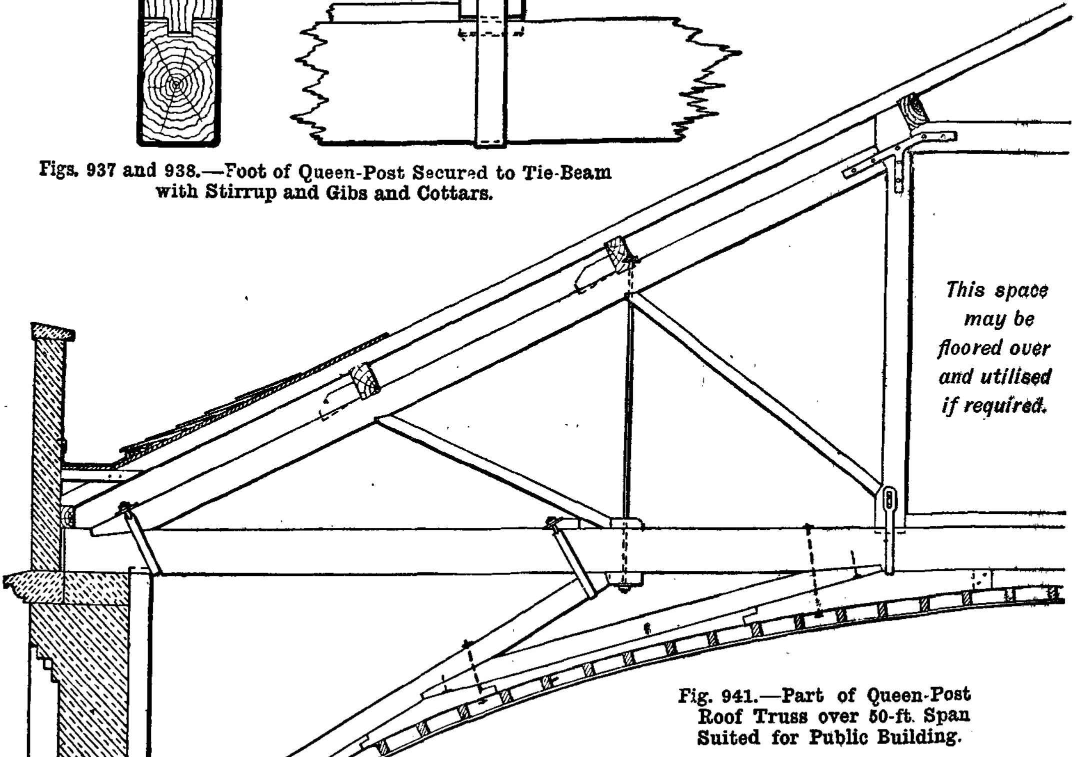

This space may be floored over and utilised if required.

Fig. 941.—Part of Queen-Post Roof Truss over 50-ft. Span Suited for Public Building.

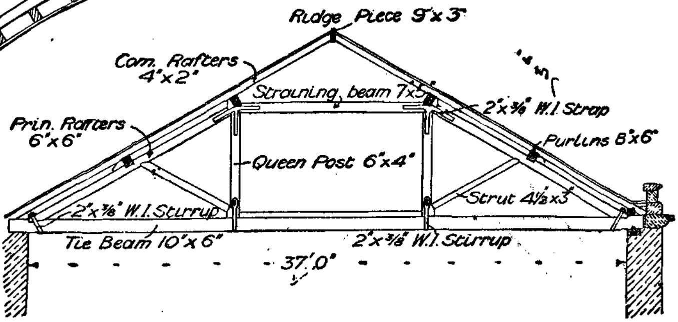

Ridge Piece 9"x 3"
Com. Rafters 4"x 2"
Straining beam 7x5
2"x⅜" W.I. Strap
Prin. Rafters 6"x 6"
Purlins 8"x 6"
Queen Post 6"x 4"
Strut 4½x3
2"x⅜" W.I. Stirrup
Tie Beam 10"x 6"
2"x⅜" W.I. Stirrup
37'.0"

Fig. 940.—37-ft. Span Queen-Post Roof Truss with Members Named and Dimensioned.

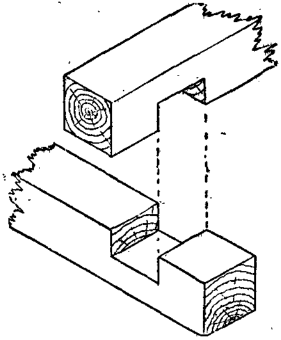


Fig. 942.—Wall Plates Double-notched Together.

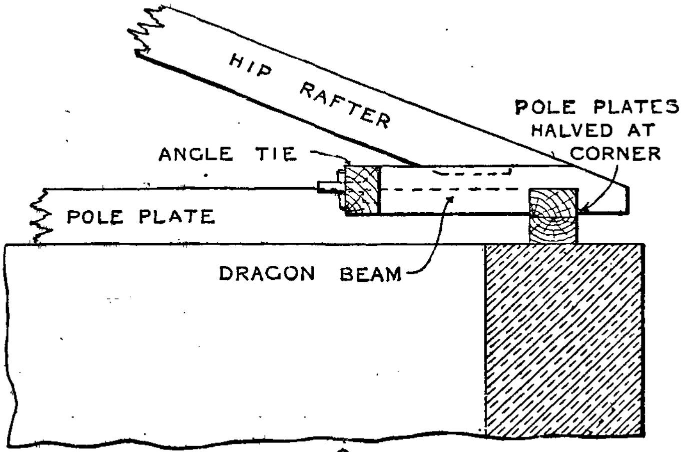

HIP RAFTER
POLE PLATES HALVED AT CORNER
ANGLE TIE
POLE PLATE
DRAGON BEAM

Fig. 946.

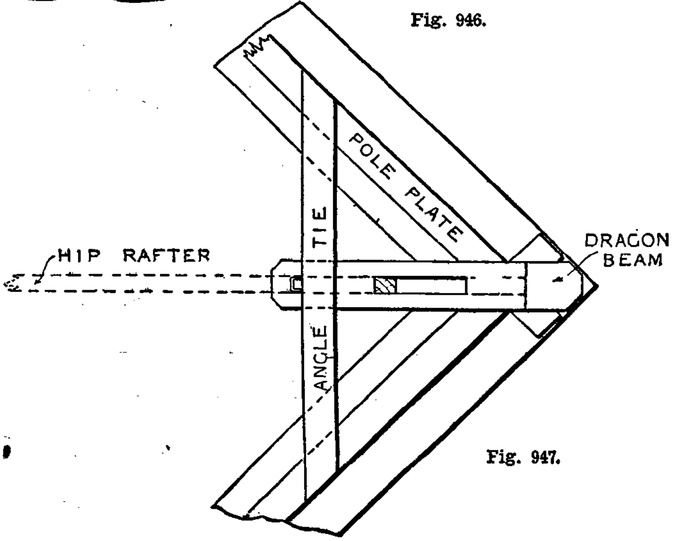

POLE PLATE
ANGLE TIE
HIP RAFTER
DRAGON BEAM

Fig. 947.

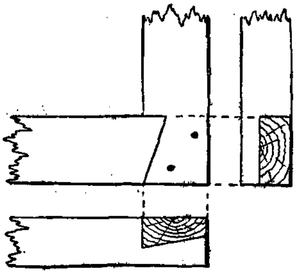
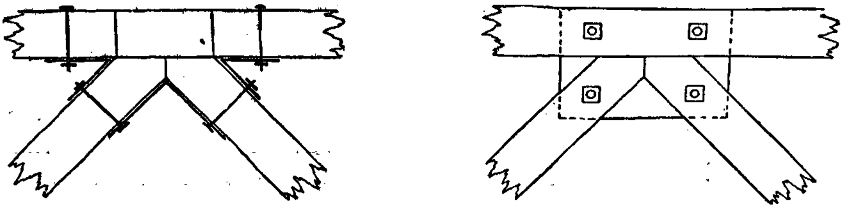


Fig. 943.—Wall Plates Dovetail-notched Together.

Figs. 946 and 947.—Vertical Section and Plan of Dragon Tie.



Figs. 944 and 945.—Members of Hipped King-Post Truss Secured Together with Straps and Plates.

Hipped King-Post Roof.

Whenever a truss is used in a hipped roof half trusses for the hips are necessary, or a dwarf queen-post truss parallel with the other trusses must be placed half way between the last truss and the end wall. Usually a half truss is applied on each hip, connecting the hip rafters at the king-post head to the principals by straps, as shown in Fig. 944, and connecting the tie-beams in a similar manner, with the addition of a plate on the bottom, as shown in Fig. 945.

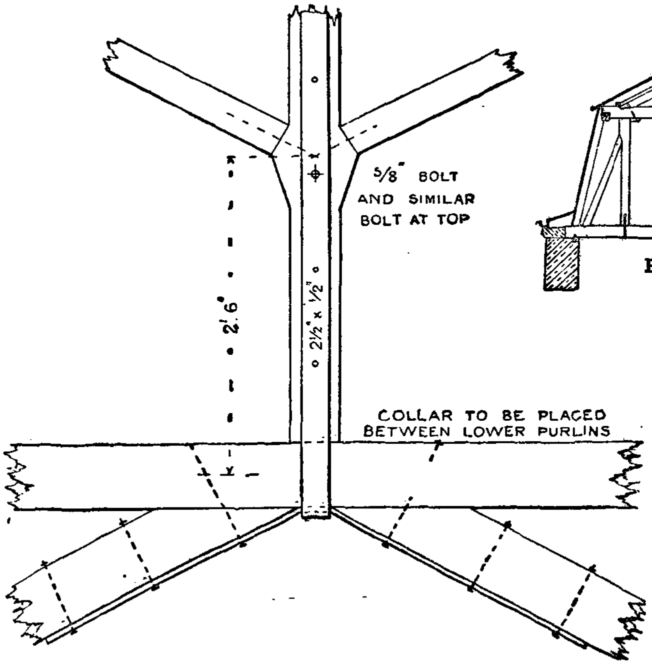

5/8" BOLT AND SIMILAR BOLT AT TOP
2' 6"
2½ x ½
COLLAR TO BE PLACED BETWEEN LOWER PURLINS

Fig. 948.—Strengthened King-Post Truss.

Dragon Tie at Foot of Hip Rafter.

A dragging tie, or dragon tie or beam (Figs. 946 and 947), is a framework at the lower end of a hip rafter connecting it with the wall plates in such a way as to resist the thrust of the hip rafter. The foot of the hip rafter is halved, notched, stepped, or tenoned into the dragging tie, which is notched at one end on to the wall-plates, at the angle where they are halved together, and at the other end is attached to the angle tie or brace by means of a tusk tenon secured by a pin or wedge, the angle tie being notched over the wall-plates to keep it in place. There is more than one method of constructing this joint. If the precise length of the hip rafter is wanted, the span of roof in section must be taken over the wall-plates only, otherwise allowance must be made for the projection of the roof covering beyond the end of the hip rafter. The dragon piece and angle tie should be used in all hipped roofs, although in small roofs it may be of a simpler construction, such as a batten nailed diagonally across the plates below hip rafter.

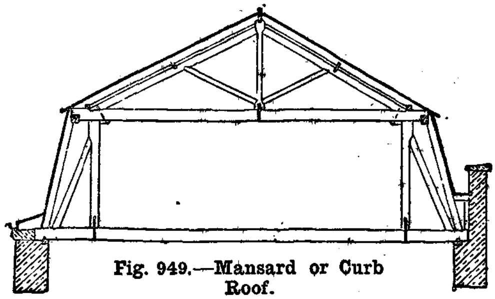


Fig. 949.—Mansard or Curb Roof.

Strengthening King-Post Truss.

The alteration of old roofs may occasionally largely increase the stresses. As an example, a church's king-post roof truss with two struts on each side may be altered by removing the horizontal tie-beams and the lower pair of struts which may have interfered with the view from the galleries. Fig. 948 shows the detail of connection at foot of the shortened king-post, which is strengthened by a bar of iron, say 2 in. by ½ in., carried up the face of the king-post on each side and passed under the strap holding the new inclined tie-beams. The lower ends of the new tie-beams should be secured to the feet of the principal rafters; they should not rest on corbels.

Mansard Roofs.

A Mansard roof, or curb roof, has each side formed with two slopes or pitches—say 60° and 30°—so that more chamber space may be available inside. They are commonly constructed as a king-post truss raised on a queen-post truss, as Fig. 950; but the same name is

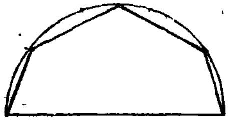


Fig. 951.

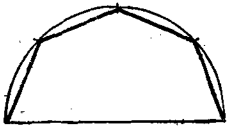


Fig. 952.

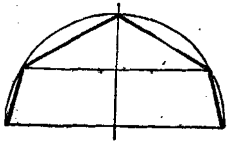


Fig. 953.

Figs. 951 to 953.—Outlines of Mansard Roofs.

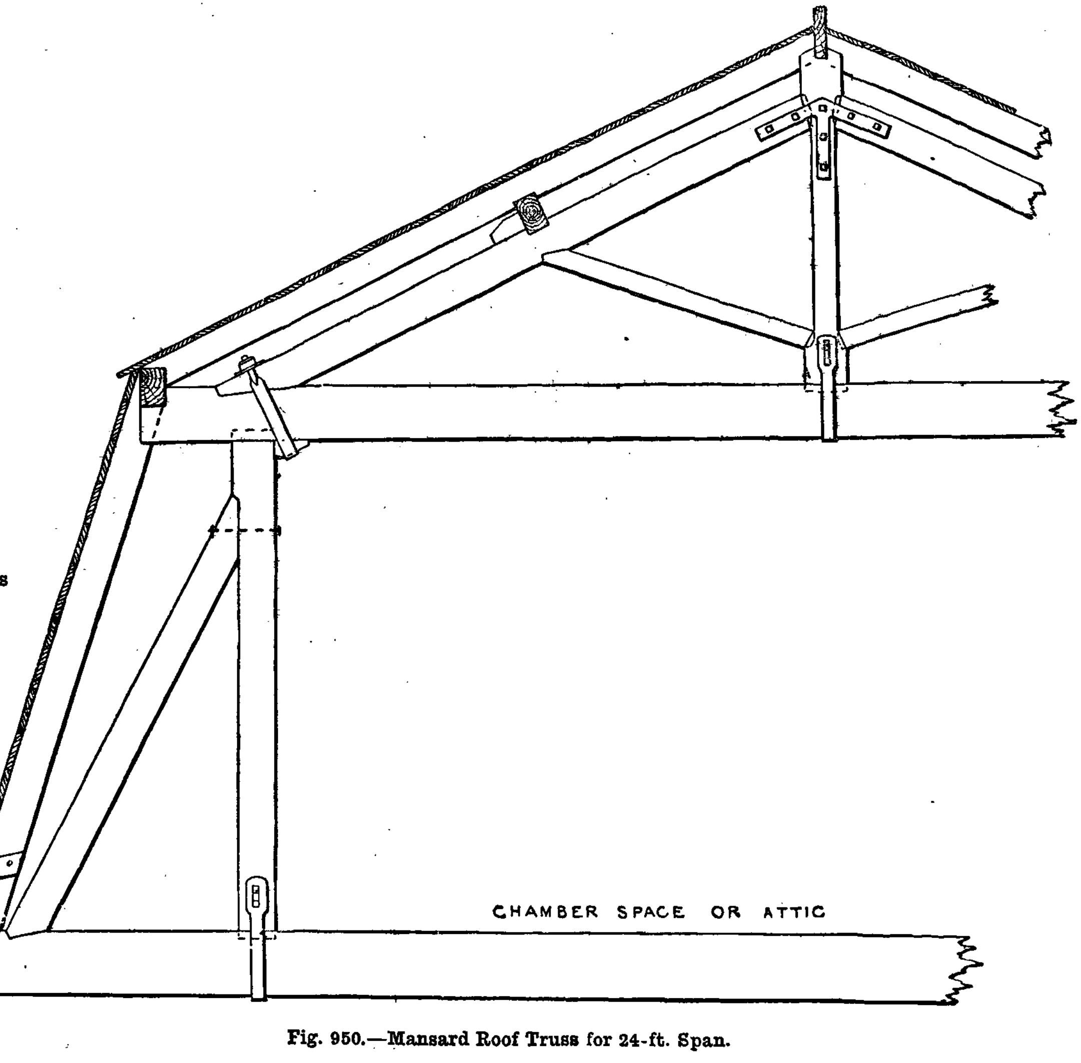

CHAMBER SPACE OR ATTIC

Fig. 950.—Mansard Roof Truss for 24-ft. Span.

retained however the internal trussing may be arranged. The construction is shown in detail by Fig. 950 (p. 238). These roofs were first used in Versailles in the seventeenth century, being designed by a French architect, François Mansard (lived 1598-1666), to make the attics available for rooms in consequence of a municipal law limiting the height of front walls. This form is largely used in the châteaux of France and, with some modification, in Germany also. It is objected to by Tredgold as being ungraceful in form and causing loss of room as compared with the original roofs of high pitch; and further, on account of the difficulty of freeing the gutters from snow. It is also dangerous on account of its inflammability. Various methods are given for obtaining the outlines of this form of roof—namely (Fig. 951), semicircle with arc divided into five equal parts, one part being taken for each lower side; (Fig. 952) semicircle with arc divided into four equal parts, one for each slope; (Fig. 953) semicircle with height divided equally to give points of curb. It is quite a matter of taste which is adopted.

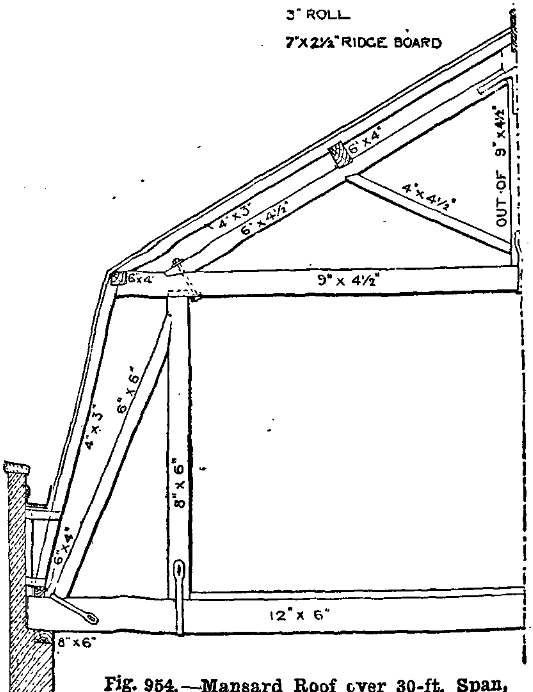


Fig. 954.—Mansard Roof over 30-ft. Span, with Parapet Wall

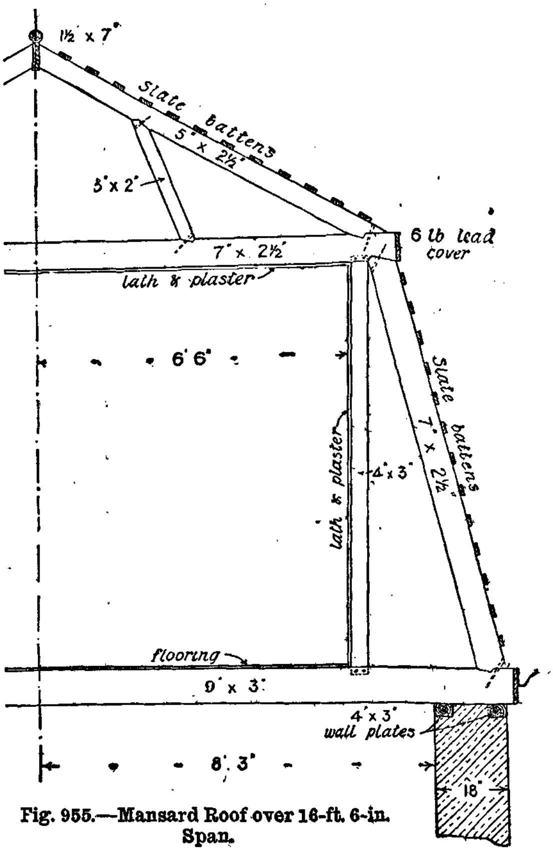


Fig. 955.—Mansard Roof over 16-ft. 6-in. Span.

Typical Mansard Roofs.

A half section of a wooden Mansard roof, over a 30-ft. span, is presented by Fig. 954, in which the scantlings of the timbers are marked, and a lead gutter and parapet wall are shown. Mansard, or curb roofs, are usually constructed with purlins and pole plates, but for a small span the purlins and pole plates may be omitted as in Fig. 955; the supports illustrated are not trusses, but trussed framings that occur at every joist and rafter, say 15 in. centre to centre.

Open Timber Roofs.

The roof truss shown in diagram by Fig. 956 is suitable for a room 80 ft. by 40 ft. Details of the various joints are given in Fig. 957. The stiffened collar-beam roof (Fig. 958), sometimes wrongly called a hammer-beam truss, is one of the various forms of open timber roofs. The scantlings that would be

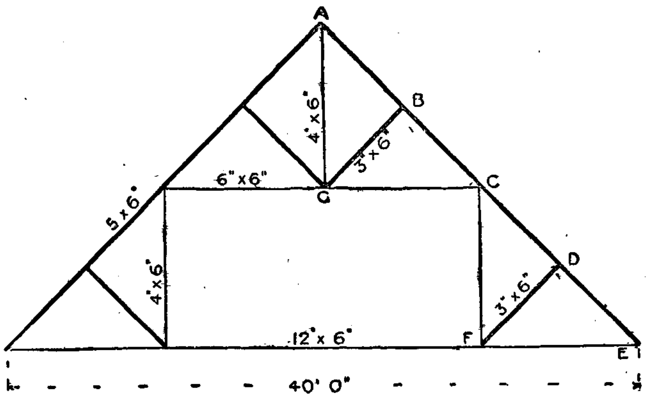

A
B
C
D
E
F
G
4"x6"
3"x6"
6"x6"
5x6"
4"x6"
3"x6"
12"x 6"
40' 0"

Fig. 956.—Dimensioned Outline of Roof Truss Suitable for 40-ft. Span

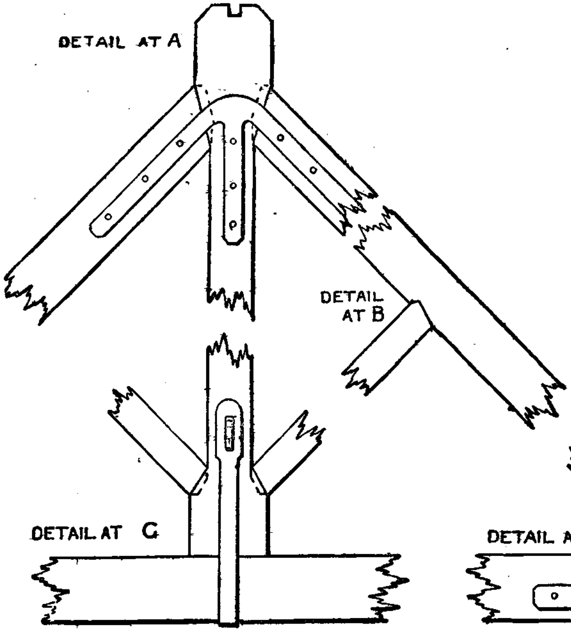
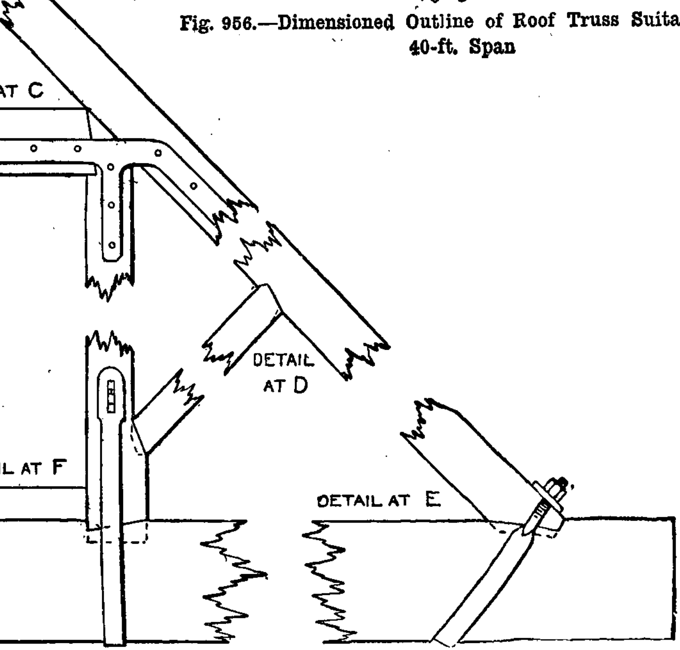
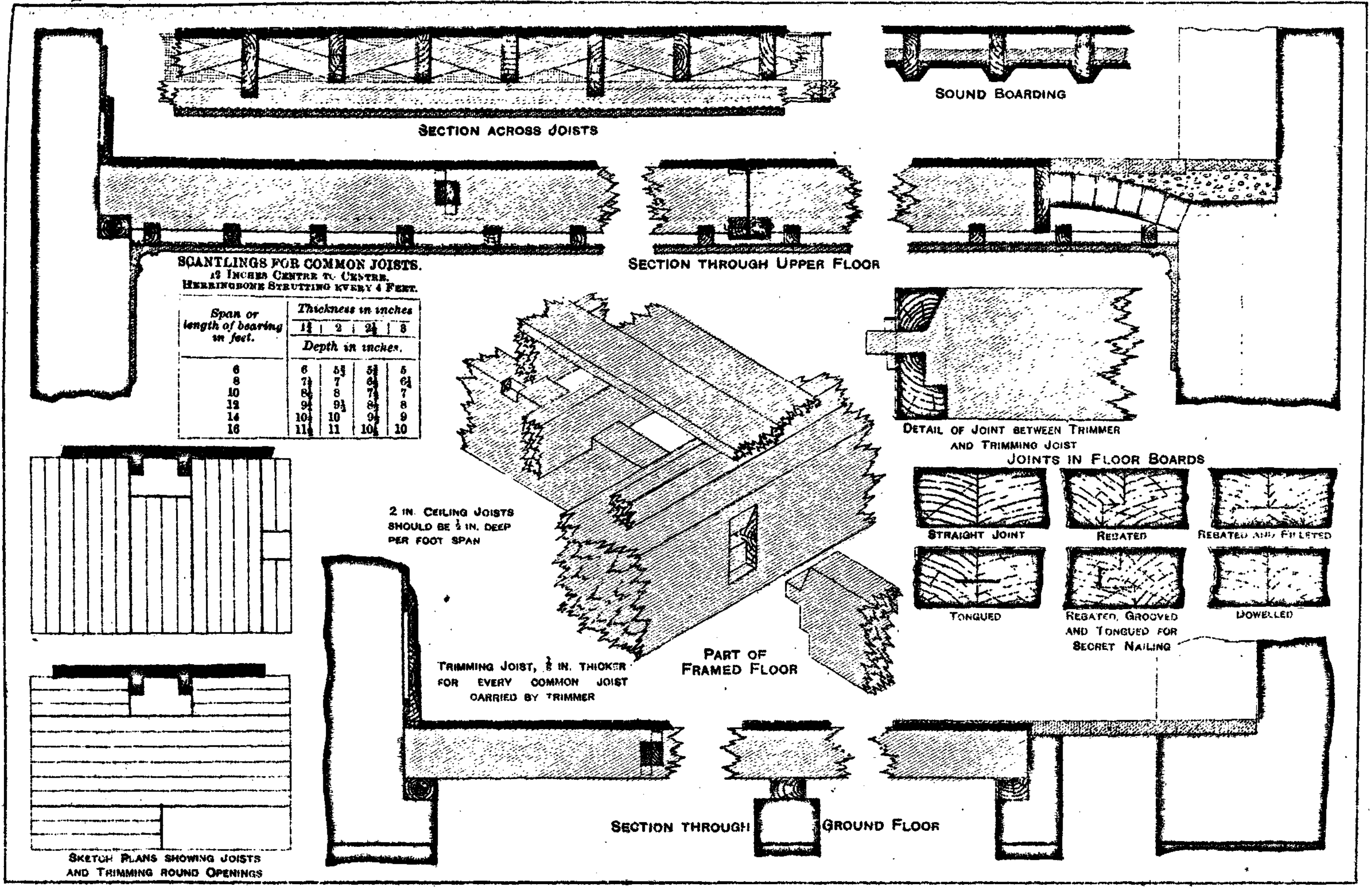


DETAIL AT A
DETAIL AT B
DETAIL AT G
DETAIL AT C
DETAIL AT D
DETAIL AT F
DETAIL AT E

Fig. 957.—Details of Roof Truss for 40-ft. Span.


SECTION ACROSS JOISTS
SOUND BOARDING
SECTION THROUGH UPPER FLOOR
DETAIL OF JOINT BETWEEN TRIMMER AND TRIMMING JOIST
JOINTS IN FLOOR BOARDS
STRAIGHT JOINT
REBATED
REBATED AND FILLETED
TONGUED
REBATED, GROOVED AND TONGUED FOR SECRET NAILING
DOWELLED
2 IN. CEILING JOISTS SHOULD BE ½ IN. DEEP PER FOOT SPAN
PART OF FRAMED FLOOR
TRIMMING JOIST, ⅛ IN. THICKER FOR EVERY COMMON JOIST CARRIED BY TRIMMER
SECTION THROUGH GROUND FLOOR
SKETCH PLANS SHOWING JOISTS AND TRIMMING ROUND OPENINGS

SCANTLINGS FOR COMMON JOISTS.
12 INCHES CENTRE TO CENTRE.
HERRINGBONE STRUTTING EVERY 4 FEET.

Span or length of bearing in feet.	*Thickness in inches*			
	1¾	2	2½	3
	Depth in inches.			
6	6	5¾	5¼	5
8	7¼	7	6½	6¼
10	8½	8	7½	7
12	9¾	9½	8¾	8
14	10½	10	9½	9
16	11½	11	10½	10

FLOORS AND FLOORING

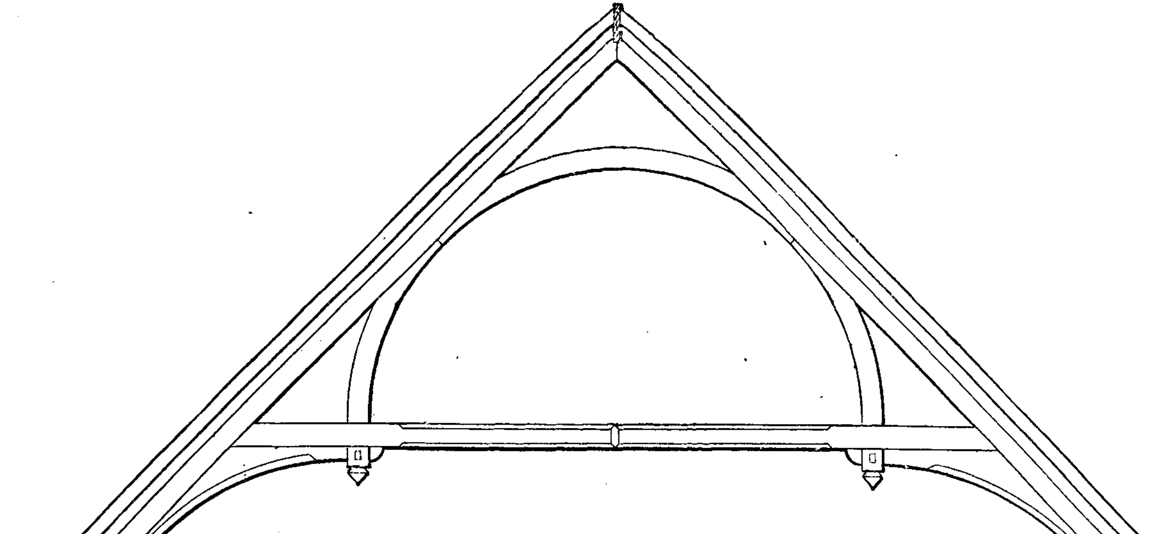


Fig. 958.—Open Timber Roof over 26-ft. Span with Stiffened Collar Beam; sometimes wrongly called a Hammer-Beam Truss.

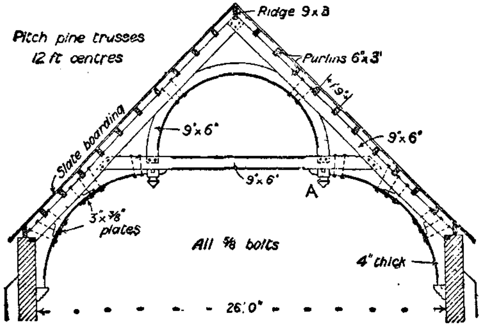

Pitch pine trusses
12 ft centres
Ridge 9 x 3
Purlins 6"x 3"
1'9"
Slate boarding
9"x 6"
9"x 6"
9"x 6"
A
3"x 3/8"
plates
All 5/8 bolts
4" thick
26'.0"

Fig. 959.—Scantlings for Open Timber Roof with Stiffened Collar Beam.

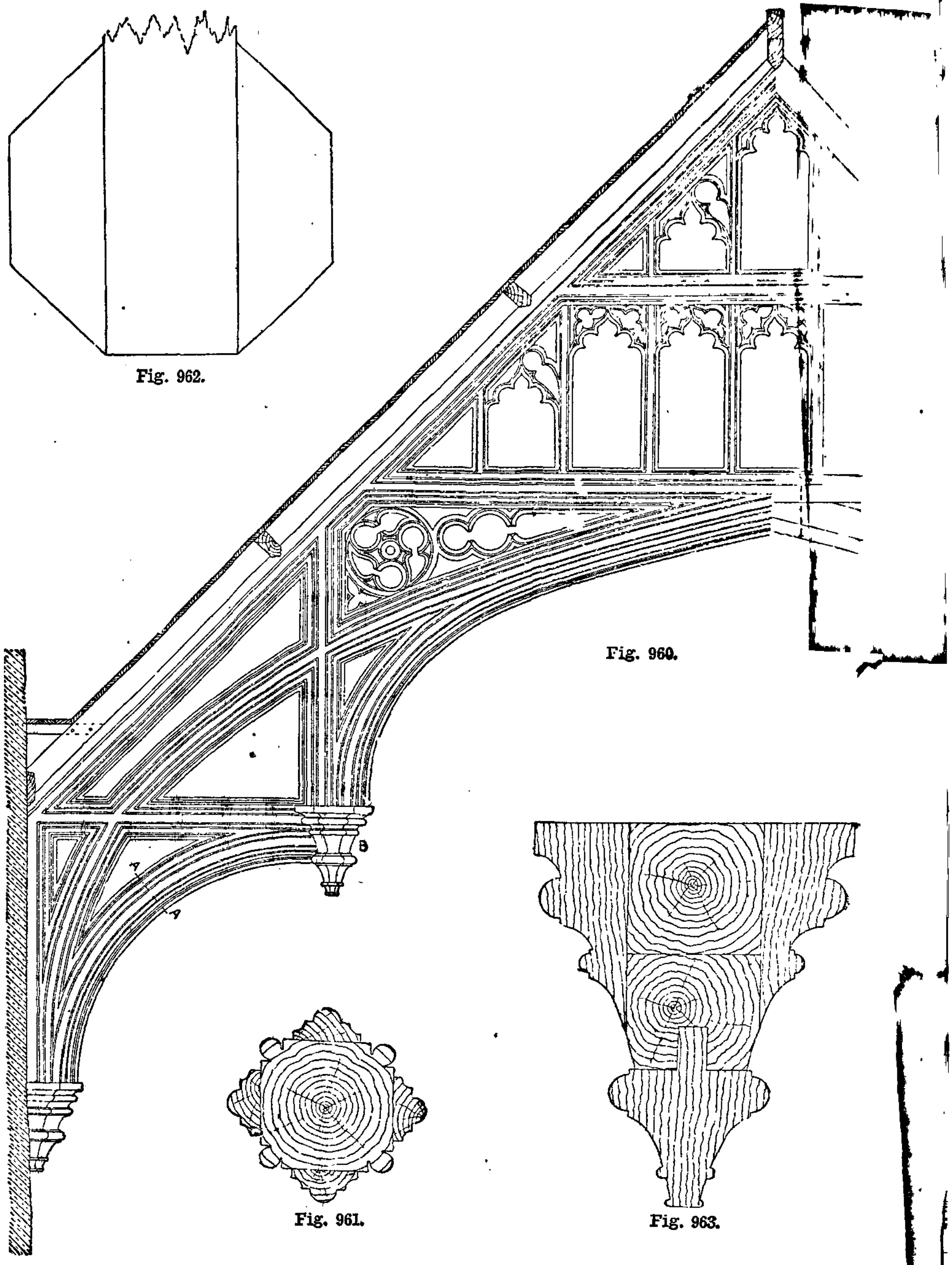


Fig. 962.
Fig. 960.
A
A
B
Fig. 961.
Fig. 963.

Figs. 960 to 963.—Hammer-Beam Truss of 50-ft. Span for Public Building with Enlarged Details.

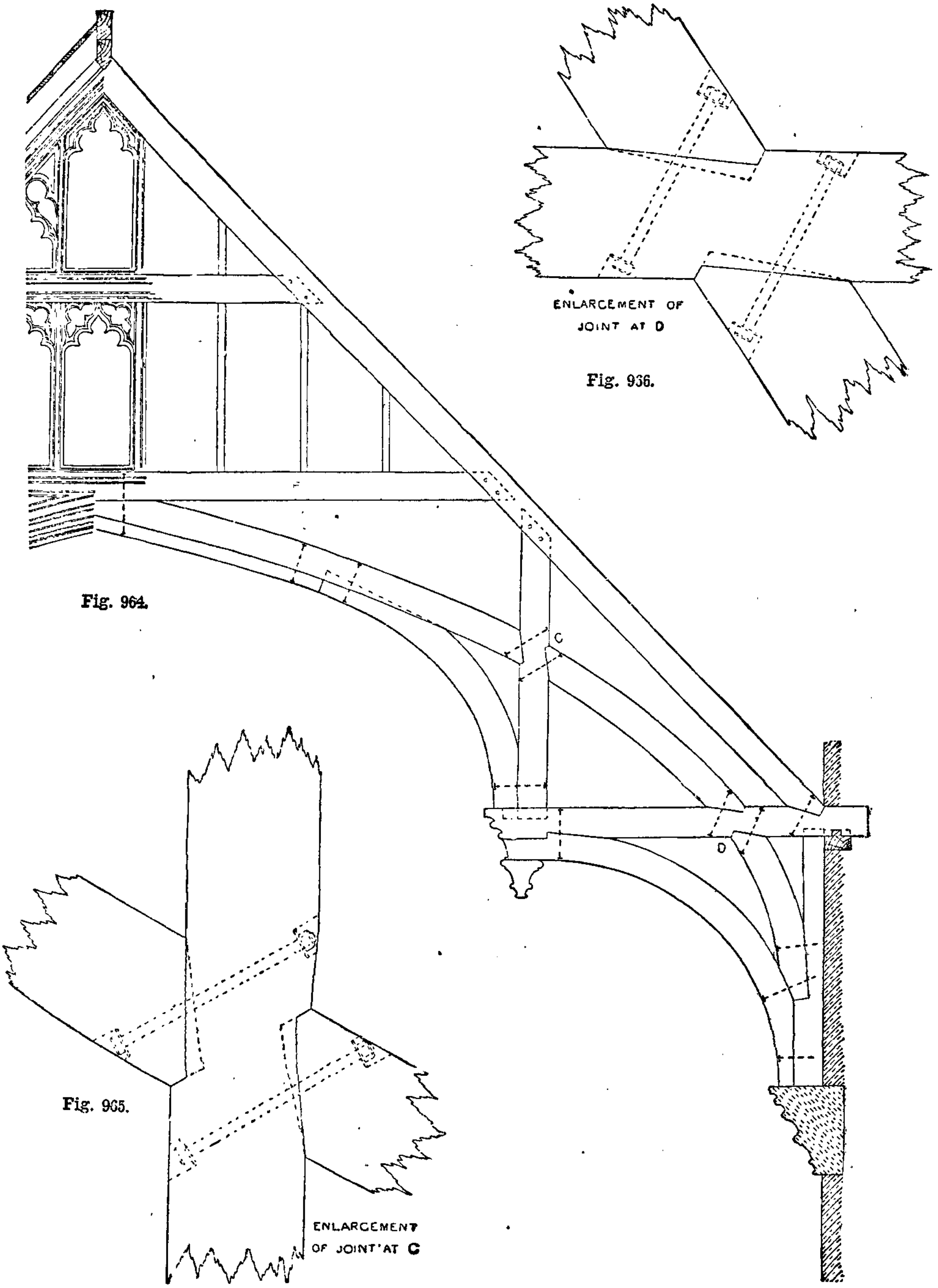

ENLARGEMENT OF
JOINT AT D
C
D
ENLARGEMENT
OF JOINT AT C

Fig. 966.

Fig. 964.

Fig. 965.

Figs. 964 to 966.—Hammer-Beam Truss showing Jointing of Members with Enlarged Details.

required for this, if the roof is covered with slating on close boarding and purlins without rafters, are shown in Fig. 959. The part marked A, which looks as though the upper semicircle is continued through the collar-beam, is only ornament, as is also the adjacent curved piece, which looks like a continuation of the curved brace. The hammer-beam truss gives the most ornamental form of open roof.

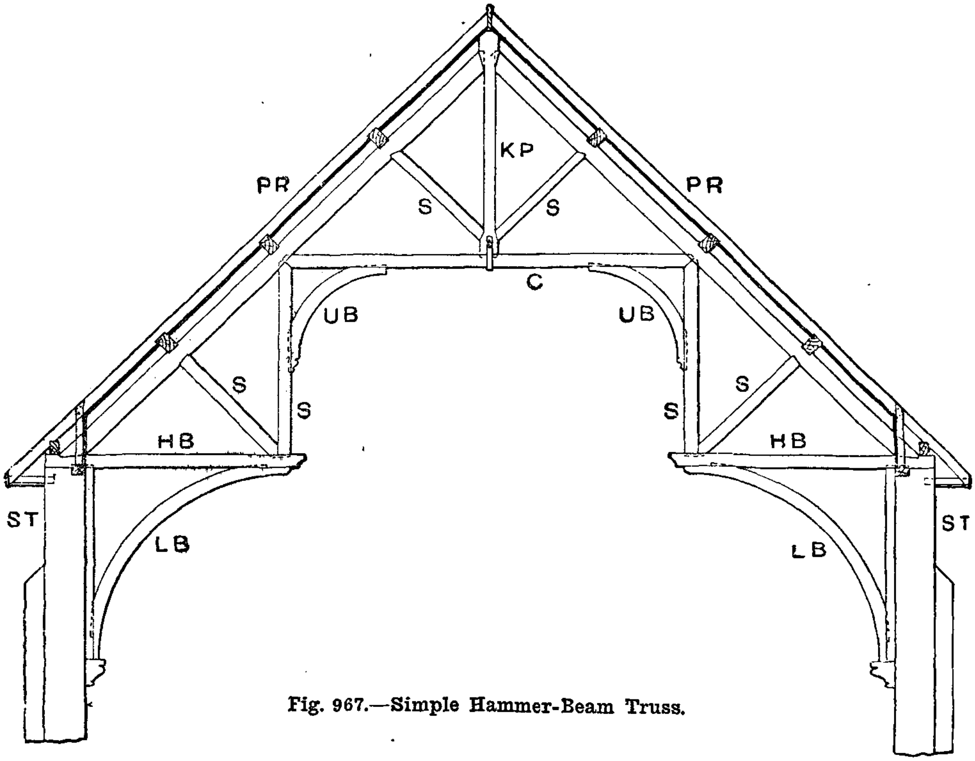


Fig. 967.—Simple Hammer-Beam Truss.

Hammer-Beam Truss.

The hammer-beam truss is a type of open timber roof, and it is shown in Fig. 967, the letters in which have the following references: P R, principal rafters; K P, king-post; C, collar; S S, struts; H B, hammer beam; U B, upper bracket or compass piece; L B, lower bracket; S T, stud. A hammer-beam truss exerts considerable thrust, and therefore substantial walls and also buttresses must be provided. A thickness of 18 in. is little enough for sound work with a span of 33 ft., but possibly the walls may be somewhat lightened by setting the window openings in 14-in. panels, and adding buttresses outside the piers. A good hammer-beam truss is illustrated on pp. 242 and 243. Fig. 960 is a half elevation of the complete construction; Fig. 961 is an enlarged section through A A (Fig. 960); Fig. 962 is a plan and Fig. 963 a section showing the detail at B (Fig. 960) to a larger scale; Fig. 964 is the half elevation of the open timber roof showing the construction without added ornament; Figs. 965 and 966 are the joints shown at C and D (Fig. 964).

Wood Lattice Roof Truss.

The wood lattice roof truss is known as a Belfast truss, and is made especially by Messrs. D. Anderson and Son, and Messrs. McTear and Co., both of Belfast. For a span of 27 ft. the lattice trusses may have a rise of 3 ft. and radius of 36 ft. and be placed 7 ft. apart (see Fig. 968). The top and bottom members may be made up by two separate thicknesses of 7-in. by 1¼-in. breaking joint. The lattice bars may be about 2½ in. by 1¼ in. and 3 ft. apart, radiating as shown. The purlins should be 3 in. by 2 in. at 3-ft. centres, and covered with ⅝-in. boarding and tarred felt. Cross bracing 4½ in. by ¾ in. between trusses as shown. The following is the rule for obtaining the radius of roof principals of the wood lattice (Belfast) pattern. If the rise be made one-tenth of the span, the radius will be thirteen-tenths of the span. Thus, 85-ft. span equals 8-ft. 6-in. rise and 110-ft. 6-in. radius, but this would be a large roof for such a system. The lattices may be arranged so that centre lines through the top and bottom apices are radial to the external curve, as shown in Fig. 969, or the lattices themselves may be drawn towards two points equal to span apart and half span below tie-beam, as shown in Fig. 970. The former has the better appearance, but the latter has more crossings where the lattices can be secured to each other to help in stiffening them. Galvanised corrugated iron forms a good covering for these roofs.

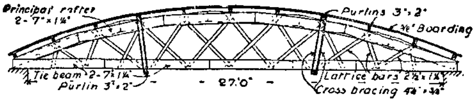


Fig. 968.—Wood Lattice Roof Truss.

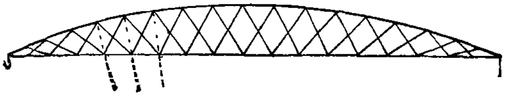


Fig. 969.—Wood Lattice Truss in Outline.

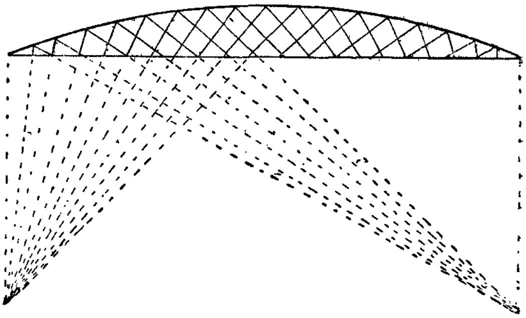


Fig. 970.—Designing Wood Lattice Truss.

Composite Trusses.

A little more than one-half of a composite roof is shown by Fig. 971. The rafters and struts are of pitchpine, and the king-bolt and ties of iron. The roof is to carry ordinary slating, and the trusses will be spaced 10 ft. apart. No holes are bored in struts or rafters; and all the ironwork is such as can be forged from the bar and fitted by a country blacksmith. The foot rests on a stone template. The bearing plate is illustrated by Fig. 972. Figs. 973 and 974 show the head and foot of the

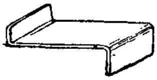


Fig. 972.—Bearing Plate for Composite Roof.

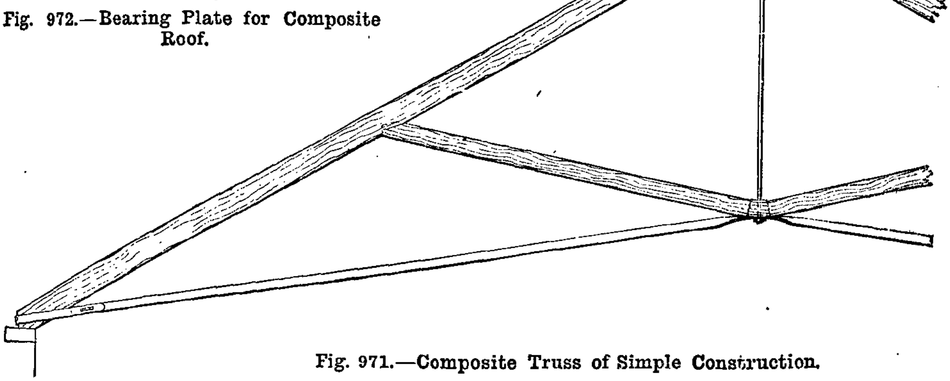
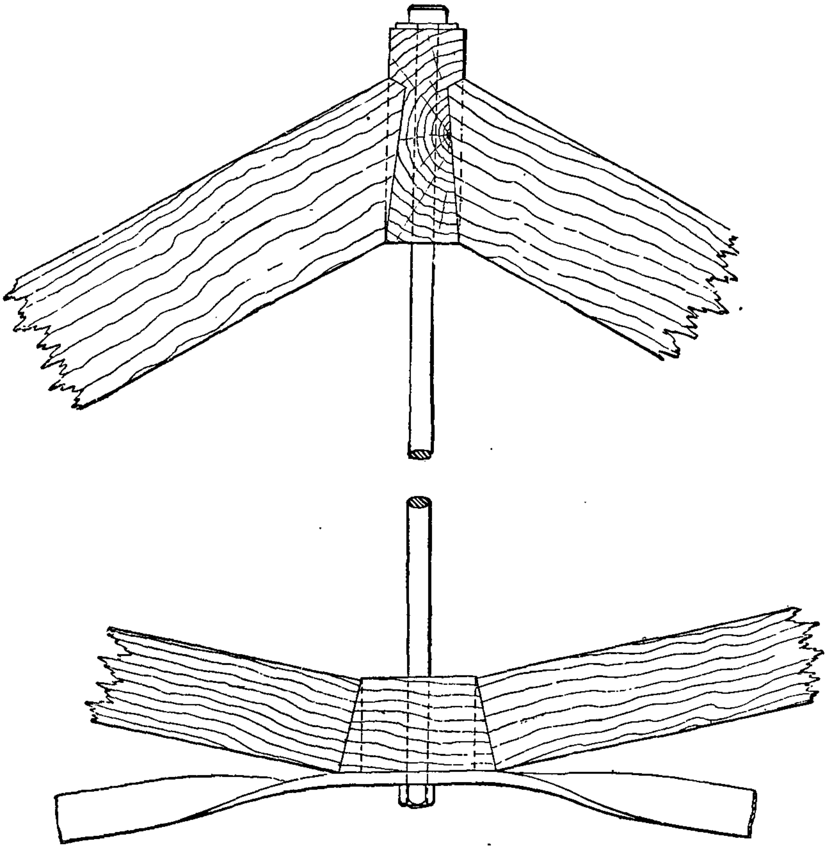


Fig. 971.—Composite Truss of Simple Construction.

king-bolt, and Fig. 975 shows the foot of the iron tie and principal rafter. Such a truss gives more head room than a wooden truss with horizontal tie-beam. It would be better with cast-iron sockets at head and foot of king-bolt. The tie rod can be adjusted to the exact length required. The truss has a lighter appearance than one all of wood. Another composite roof truss is shown by Fig. 976, this being suitable for a span of about 34 ft. A little more than half of the section of a timber roof over a 40-ft. span is illustrated by Fig. 977. The roof has a lead flat at top, with slated side slopes. The trusses are taken as at 12-ft. centres, ceiled to the underside of the tie-beams, and resting on rubble walls lined with brick. A lead roll is shown at the ridge, and a cast-iron ogee gutter, with fascia and soffit boards, at the eaves.



Figs. 973 and 974.—Head and Foot of King-Bolt of Composite Wood and Iron Roof Truss.

Arched Ribs Pivoted at Springings and Crown.

Arched ribs are sometimes pivoted at the springings and the crown, because then the line of thrust is compelled to pass through these points, and it is easier to determine the stresses. These arched principals are only used in very large spans, and then generally for architectural effect, or for allowing a clear headway for any purpose. Their stability depends upon the stiffness of the ribs themselves, and also upon the suitability of the supports to resist a lateral pressure.

Roof Truss Over Octagonal Apse.

A semi-hexagonal apse having three full sides is easier to roof than a semi-octagonal one, as it requires only two half-trusses. In

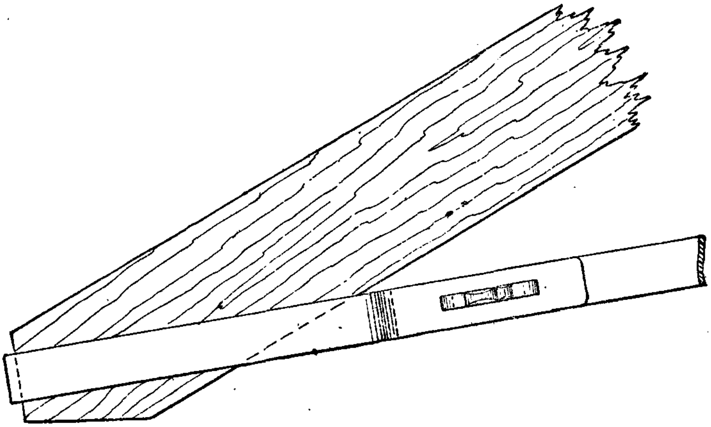


Fig. 975.—Foot of Rafter of Composite Wood and Iron Roof Truss.

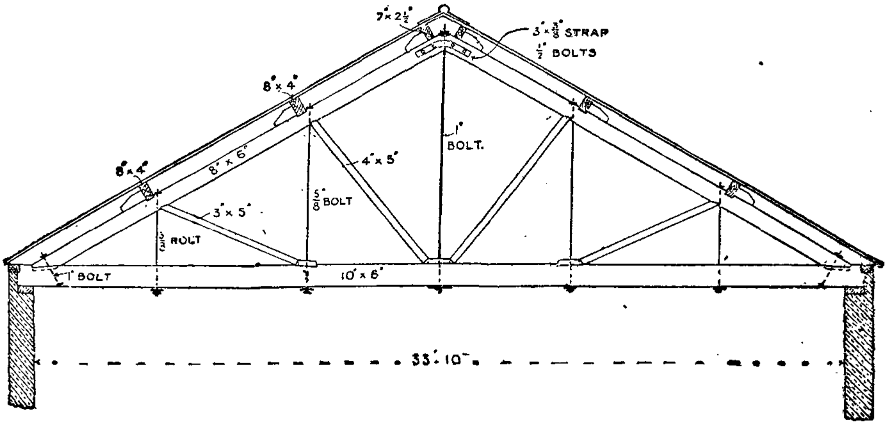

7" × 2½"
3" × ⅜" STRAP
½" BOLTS
8" × 4"
8" × 4"
8" × 6"
1" BOLT.
4" × 5"
⅝" BOLT
3" × 5"
½ BOLT
1" BOLT
10" × 6"
33' 10"

Fig. 976.—Composite Roof Truss for Span of 35 ft. to 45 ft.

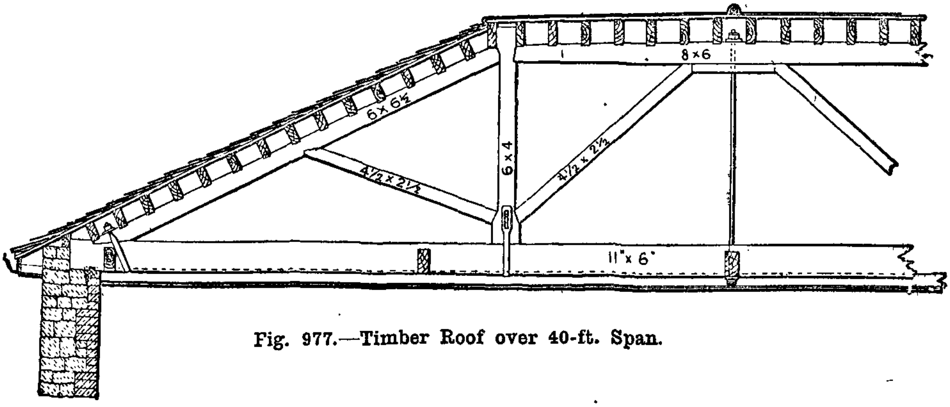
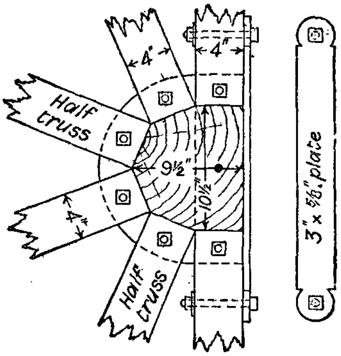

8 × 6
6 × 6½
6 × 4
4½ × 2½
4½ × 2½
11" × 6"

Fig. 977.—Timber Roof over 40-ft. Span.

the case of an octagonal apse the king-post, having to receive the members of its own truss and four half-trusses besides, must be of larger section and shaped as in Fig. 978, so that the principal rafters and upper braces may be framed into it. There should be a wrought iron plate bolted on the side of the main collar to stiffen it against the half-collars, and this may be shaped as shown in the elevation (Fig. 979). The king-post may be halved at the foot or to the collar of the main truss, as in Fig. 980, and the half-collars connected by tenons and bolted through a wrought-iron plate as shown in the plan of the underside at Fig. 981.



Figs. 978 and 979.—King-Post for Roof Truss over Octagonal Apse.

High-pitched Roof Truss.

A roof 18-ft. span with a height equal to the span will present a large surface to the wind and require strong walls and stout scantlings of timber. A king-post truss as shown in Fig. 982 will be suitable, the scantlings being as follows: Tie-beam 12 in. by 6 in., king-post 6 in. by 6 in., head and foot of king-post 12 in. by 6 in., principal rafters 6 in. by 6 in., struts 3 in. by 6 in., purlins 9 in. by 6 in., common rafters 5 in. by 2 in., trusses 10 ft. apart, with all joints very securely fixed. A little saving of timber might perhaps be effected if the truss were braced as shown in Fig. 983; but this would be rather an unusual form of truss.

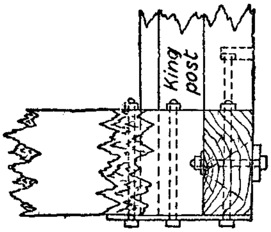


Fig. 980.—Joint of King-Post and Collar.

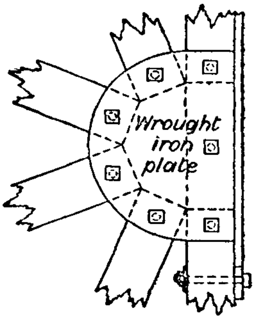


Fig. 981.—Wrought-Iron Plate on King-Post.

Determining Lengths and Bevels of Rafters and Purlins.

Length of Valley Rafters.—In the four following paragraphs it is assumed that the roofs are of different pitch. In Fig. 984, the method is shown of obtaining the lengths of valley and jack rafters. Begin on the plan of the roof by drawing *b c* at right angles to valley rafter *a b*, making *b c* equal height from wall plate to ridge; then join *a c*, which is the true length of the valley rafter.

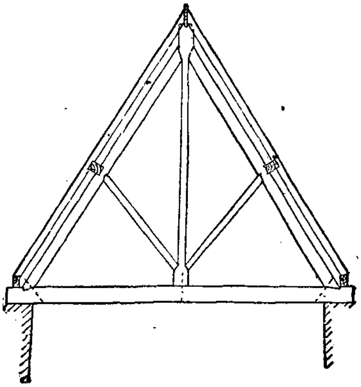


Fig. 982.—High-pitched Roof Truss.

Length of Jack Rafters.—To find the length of jack rafter *d e*, draw *f g* at right angles to wall plate, and set up *g h* equal to the height of the roof *b c*; join *f h*, and from *d* drop perpendicular to *d e* to cut *f h* in *o*, then *o h* is the true length. Fig. 985 shows the method of obtaining the cuts on the valley rafter at junctions with ridges and wall plates. First join intersection of ridges with the outside point where the wall plates intersect (*s*). Then draw a line equal to the width of the valley rafter square across the plan of the valley, to cut each ridge in the edges of the valley. This line will cut the same distance along each ridge when both roofs are of the same pitch, but not otherwise. The side elevation is taken at the side *a b*, and projected upon a plane parallel to it marked *c d*. The pitch of elevation lines for the valley rafter is taken from the angle *b a c* (Fig. 984). The plumb cut notching on the wall plate and the depth are shown on the side elevation. The piece sectioned at the top end of the rafter is where it butts against the ridge.

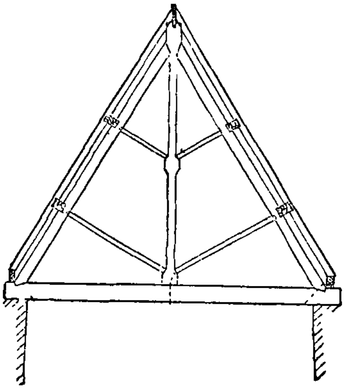


Fig. 983.—High-pitched Roof Truss.

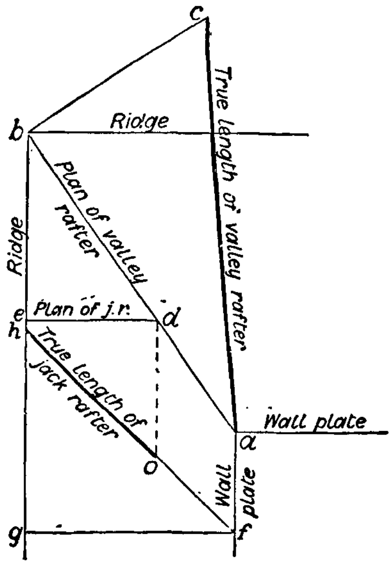

c
b
Ridge
True length of valley rafter
Plan of valley rafter
Ridge
e
h
Plan of j.r.
d
True length of jack rafter
o
Wall plate
a
Wall plate
g
f

Fig. 984.—Obtaining Lengths of Rafters.

Bevels of Valley Rafters.—To find the bevels, develop the edge, and project each point as shown. For the top end it will be seen that the centre of the valley does not run into the corner. This is owing to the two different pitches. Set off the same distances shown in plan *c f*, make *e x* on the edge equal to *e x* in plan, and join *e a* and *e f*, and the bevels to fit the ridges are determined. To make the bottom end of the rafter fit the wall plates, the points are taken in a similar way, each point being projected from the plan. For the inside the points are *n o p*, and for the outside *r s t*. It will be seen that the points *o*, *s* inside and outside are not the same distance from the edge.

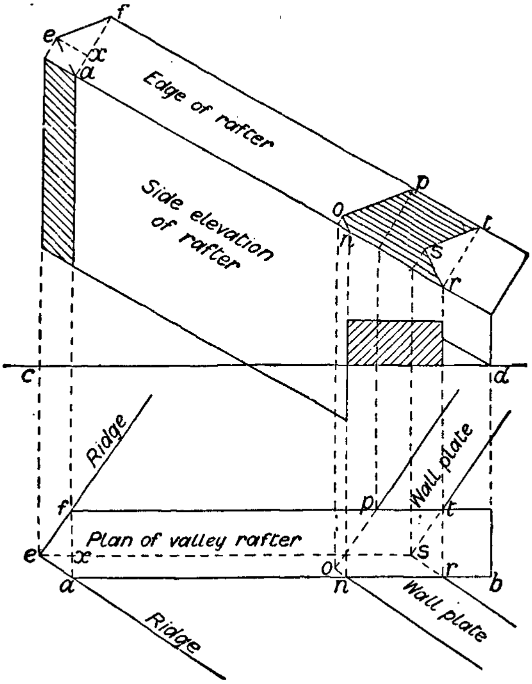

f
e
x
a
Edge of rafter
Side elevation of rafter
o
p
t
s
n
r
c
d
Ridge
Wall plate
f
p
t
Plan of valley rafter
e
x
o
s
r
b
a
n
Ridge
Wall plate

Fig. 985.—Obtaining Cuts on Valley Rafter at Junction with Ridges and Wall Plates.

Bevels of Jack Rafters.—Fig. 986 shows the method of obtaining the bevels of the jack rafters or short spars. To find the side cut for the jack rafter A, first set up the pitch of the jack rafter as *a b c* by method shown in Fig. 984. Now drop a perpendicular from the point *d* in the plan until it meets *b c* in *e*; next set up a line from *c* at right angles to *b e c*, and cut off at *f* equal to depth of jack rafter. Through *f* draw a line parallel to *b e c* until it meets *d e* in *g*; then *g h c e* is the side elevation of jack rafter A, and *g e c* and *e c a* are respectively side cuts at junction with valley rafter and ridge. Now drop perpendicular from *m* to *n*, and then square with *g n f*; set up *n o* equal to the width of the jack rafter on plan. Join *o g*, then *o g n* is the edge cut where the jack and valley rafters meet. In the same manner the cuts for the jack rafter B may be found as shown.

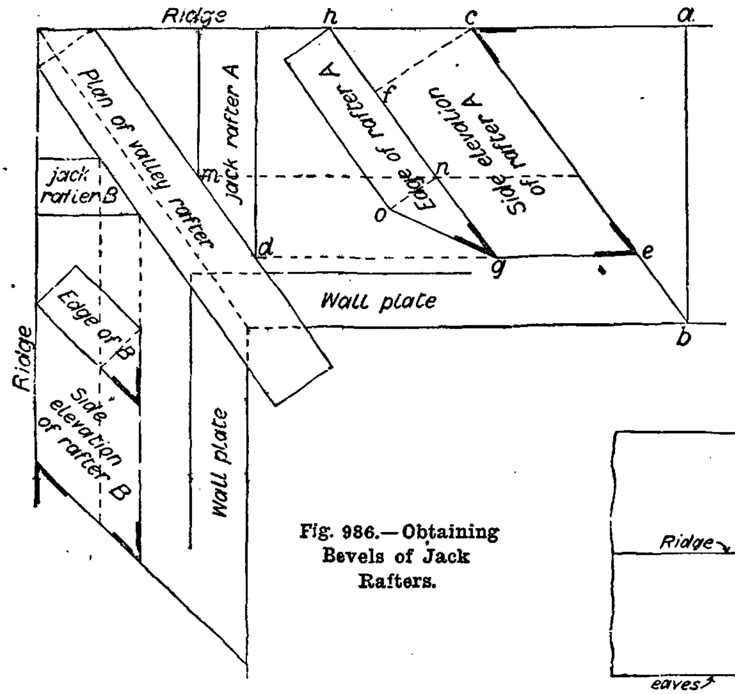


Fig. 986.—Obtaining Bevels of Jack Rafters.

Finding Length and Bevel of Valley Rafter: Another Case.—It is required to find the lengths and bevels of two valley rafters where one roof joins another. The arrangement of the roofs is shown in Fig. 987, both roofs being of the same pitch but of different heights. Line *a b* represents the plan of the valley; set up *b c* equal to the rise of the roof, and join *a c*. Then *a c* will be the length of the valley rafter, having the angle *b a c*. Fig. 988 is the plan of the valley rafter, wall plates and ridges. Fig. 989 is an elevation projected on a parallel plane to the side *a b* of the valley rafter, showing at the top end *d* in section lines the bevel required to fit the rafter *c*, and at the bottom *e* the bevels required to fit over the wall plates. Fig. 990 shows the method of finding the angle at the back of the valley rafter. At any point *f* in *d e* draw the line *g h* square to *d e*, and produce the two wall plates to meet this line; make *e i* square to *e d* and equal to the rise of the roof, join *i* to *d*, and the length and inclination of the valley is shown. From *f* draw *f k* square to *i d*, and turn the line *f k* upon *e d* from *f* to *m*, and join *m* to *g* and *h*, and the true shape of the angle is obtained.

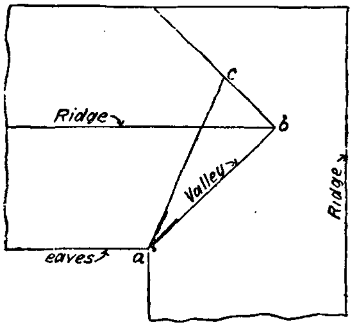


Fig. 987.—Two Roofs of Same Pitch but Different Heights.

Determining Angle of Hip Rafter.—The following is a method of finding the angle at the hip of a roof; for example, the intersection of the main roof (37½°) at the eaves, with a hipped end (45°) at the eaves. The hipped end of a roof is not usually laid at a different angle from the sides; but in the above example, where the main roof is at 37½°, and the hipped end 45°, the angle of the hip rafter will be found as follows: Draw a plan of the end of the building *a b c d* (Fig. 991) and section *e f g* (Fig. 992) with point *e* exactly opposite point *c*; then the order of lettering shows the order of construction. Using the section now as the elevation of the roof towards one end, set up *e b* at the angle corresponding to the hipped end and draw *g h*. Bisect the plan by a line through *i*, to represent the ridge and produce the line vertically from *h* to *i* in order to give the termination of the ridge and the intersection with the hips. Join *i c*, *i b* to show the plan of the two hip rafters. From *i* set up the right angle *c i j* = 90°, cut off *i j* equal to the height of the roof *e k* and join *j c*. Then *j c i* will be the true angle of the hip rafter with the length *c j*.

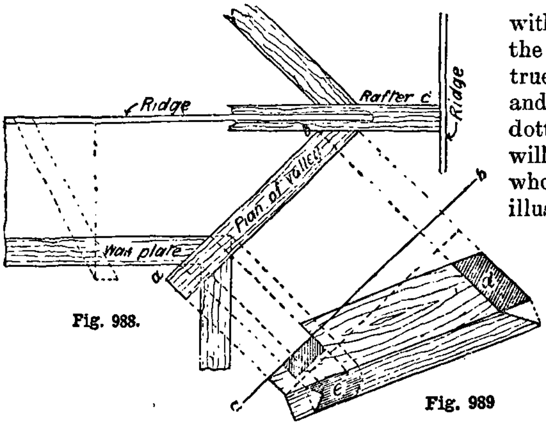


Fig. 988.

Fig. 989

Figs. 988 and 989.—Obtaining Length and Bevel of Valley Rafter.

Lengths and Bevels for Rafters of Lean-to Roof.—In the case of a lean-to roof where the width of the span varies from one end to the other, either the ridge or the eaves must be out of level or the plane of the roof must twist. In the latter case each rafter would require a different bevel. A practical carpenter would in such circumstances probably try each rafter in place and scribe the line of cut, which would be quicker than setting off the bevels geometrically. Another method is to keep the sloping roof parallel, and have a lead flat tapering from the width of the gutter at one end to the whole difference of the span and girder at the other end; this is the more usual method with factories and warehouses, where the ground plan is often irregular and no space can be thrown away.

Bevels for Purlins.—The following shows how to get the bevels of purlins against hip rafter, the purlins being square from the roof. Fig. 993 shows a section through the hipped end of an ordinary roof, and Fig. 994 shows the plan, with one hip and one purlin in position. To the right and left of Fig. 994 are shown the true views of the purlin cut to fit the hip rafter and to meet the other purlin below it. The dotted projection lines from section and plan will show how these cuts are obtained. To one who has studied drawing and projection this illustration will not present difficulty, but the usual method of finding the bevels is shown in Fig. 995, where the purlin is cut straight down the face of the hip, and does not meet the other purlin. The order of working is shown by the order of lettering. Another very simple method is shown by Fig. 996, the order of working being

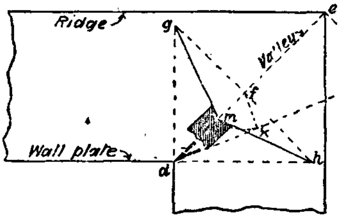
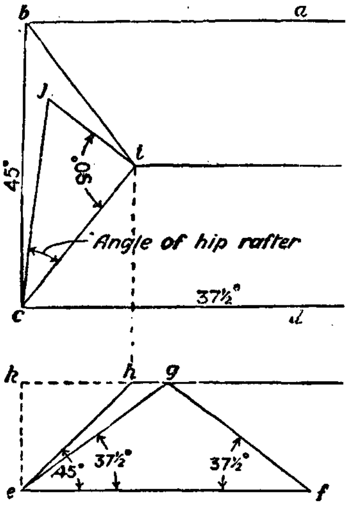


Fig. 990.—Obtaining Angle at Back of Valley Rafter.

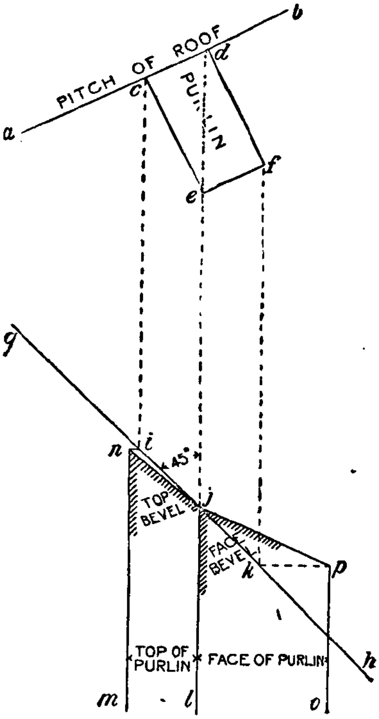


Figs. 991 and 992.—Obtaining Angle of Hip Rafter.

shown by the lettering. From the underside of purlin drop verticals *a b*, across these draw the square line *c d*, from *d* with the length *a b* cut *a c* in *e*, from *c* with radius *c f* tangent to *d e* draw arc *f g*, join *d g*, then *d g c* will be the top bevel. From *d*, with radius *d e*, strike arc *e h*, join *c h*, then *h c d* will be the face bevel.

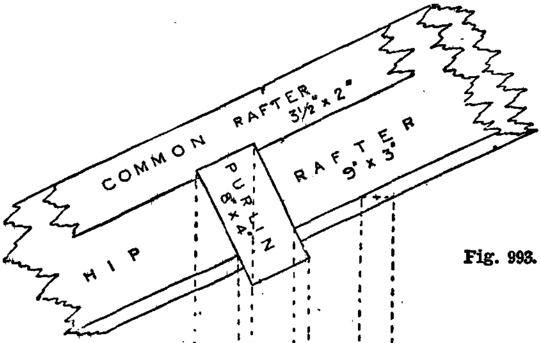
Junction of Two Roofs.

Assume a case of a main roof being 9 ft. high, while that of an addition is only 8 ft. It is required to get a bearing for the ridge and valley boards connecting the addition to the main roof, as one ridge will be 1 ft. below the other. There are various methods of dealing with this case. (*a*) If the rafter of the main roof run through in way of the addition, the ridge of the addition will simply meet one of the main rafters, and the valley rafters carrying the jack rafters will lie upon the main rafters on each side, and the whole may be stiffened by horizontal fishing pieces on each



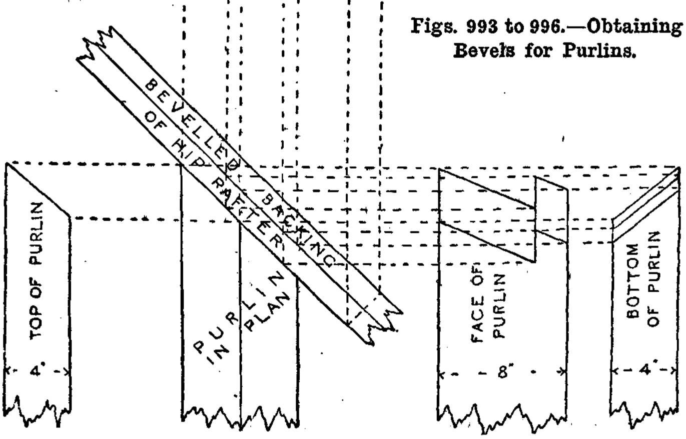
b
PITCH OF ROOF
d
c
a
PURLIN
f
e
g
n
i
45°
j
TOP BEVEL
FACE BEVEL
k
p
h
TOP OF PURLIN
FACE OF PURLIN
m
l
o

Fig. 995.



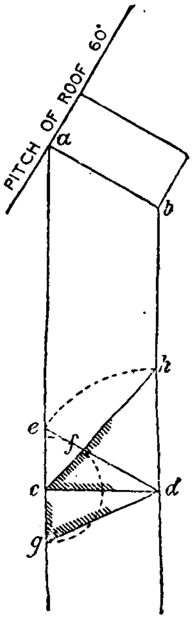
COMMON RAFTER 3½" x 2"
PURLIN 8" x 4"
RAFTER 9" x 3"
HIP

Fig. 993.



BEVELLED BACKING OF HIP RAFTER
TOP OF PURLIN
4"
PURLIN IN PLAN
FACE OF PURLIN
8"
BOTTOM OF PURLIN
4"

Fig. 994.



PITCH OF ROOF 60°
a
b
h
e
f
c
d
g

Fig. 996.

Figs. 993 to 996.—Obtaining Bevels for Purlins.

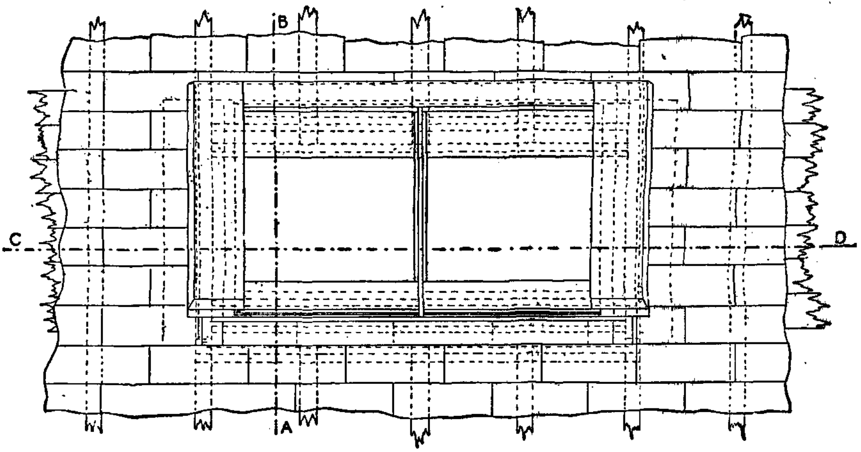


Fig. 997.

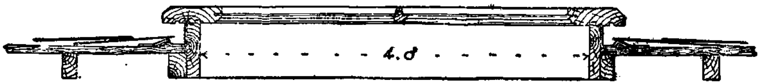


Fig. 998.

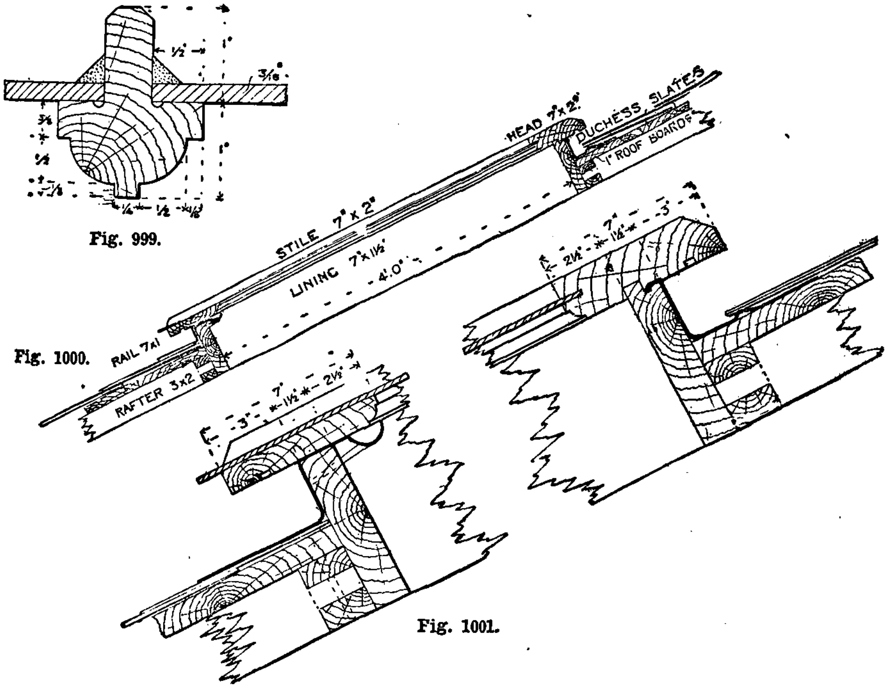
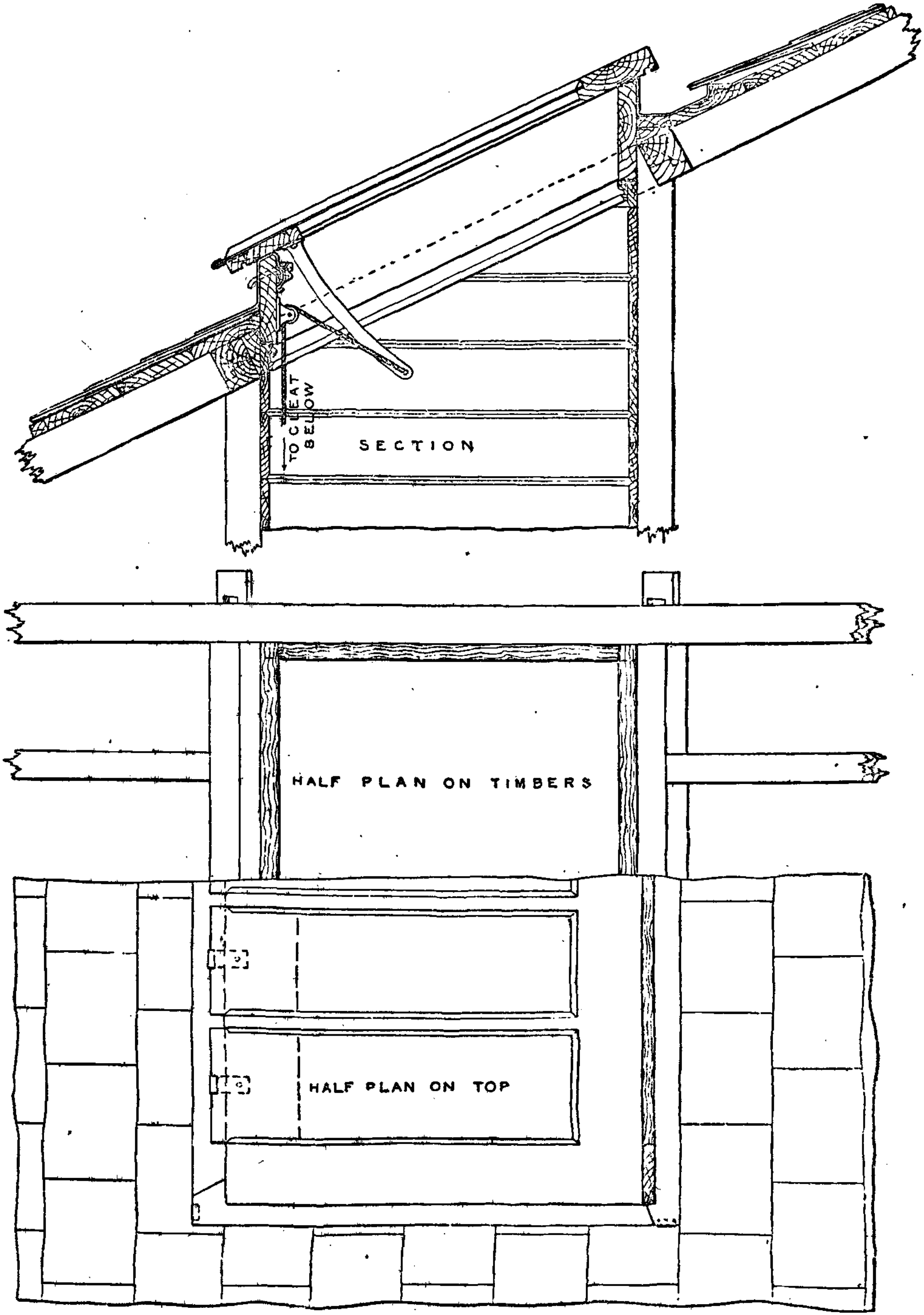

½"
1"
3/16"
¾
¼
⅛
¼
½
⅛
1"
HEAD 7"x2"
DUCHESS SLATES
1" ROOF BOARDS
STILE 7"x2"
LINING 7"x1½"
4'.0"
RAIL 7x1
RAFTER 3x2
3"
7"
1½"
2½"
2½"
7"
1¼"
3"

Fig. 999.

Fig. 1000.

Fig. 1001.

Figs. 997 to 1001.—Skylight in Slated Roof.


TO CLEAT BELOW
SECTION
HALF PLAN ON TIMBERS
HALF PLAN ON TOP

Figs. 1002 and 1003.—Skylight to Open in Slated Roof.

side of the lower ridge, and across the pair of main rafters meeting at that point. (*b*) If the roofs are open, or the main rafters do not run through, the lower ridge would run through to meet the rafter on the other side of the main roof, and be held by iron straps or wood fishplates. The valley rafters would then meet at the lower ridge and receive the jack rafters from the main roof, as well as from the addition. A short rafter would then run from the upper ridge to the lower ridge on the centre line to complete the main roof.

Life of Wooden Truss.

The life of a wooden roof truss depends entirely on whether the wood is well seasoned and so fixed as to be ventilated and protected from moisture. In such circumstances a fir truss may last a hundred years; oak has been known to last for several centuries and still be sound. The most notable example is perhaps that of the fir trusses of the old part of the roof of the Basilica of St. Paul at Rome, which were framed in A.D. 816, and were sound and good in 1814, a space of nearly a thousand years.

Skylights in Slated Roofs.

A plan and section of a common skylight in a slated roof are shown by Figs. 997 and 998, the sash-bar being shown in section by Fig. 999. A section and enlarged details of a fixed skylight are given by Figs. 1000 and 1001. A plan and section showing a 4-ft. by 3-ft. skylight, designed to open in a slated roof having a slope of 1 to 2, are presented by Figs. 1002 and 1003. It may be at the top of a shaft of the size of the skylight frame. All the details of construction are shown clearly, including the leadwork, the hinges, and the means by which the light can be easily opened to a varying extent by a person in the room below.

Measuring Carpenters' Work. Some Terms Explained.

"**Fir fixed**" is the term applied to timbers which are simply cut to length and nailed to other timbers.

"**Fir framed**" is applied to timbers requiring any labours other than described for fir fixed, such as tenoning, housing, mitering, cogging, notching, etc. It includes fixing.

"**Fir framed in trusses**" is applied to those parts of roofs and partitions which form properly braced trusses, independently of added parts included in the above items.

In an ordinary king-post roof, fir fixed includes the wall plates and cleats; fir framed includes the rafters, purlins, ridge, hips and pole plates; fir framed in trusses includes the principal rafters, king-post, braces, and tie-beam.

Example of Quantities for Roof.

Take for example 30-ft. run of a king-post roof 20-ft. span, pitch 30 degrees, trusses 6 ft. apart, principal rafter 4 in. by 4 in., tie-beam 9 in. by 4 in., king-post 4 in. by 4 in., struts 4 in. by 3 in., purlins 6 in. by 4 in., rafters 4 in. by 2 in., ridge piece 7 in. by 2 in., pole plates 6 in. by 4 in., wall plates 4½ in. by 3 in.

The Dimension Sheet will be as follows :—

	ft. in.	*ft. in.*	*ft. in.*
			20 0 2/9= 1 6 21 6
5/	21 6 — 4 — 9	26 11	Fir frd. in trusses. Tie-beam.
5/2/	11 6 — 4 — 4	12 10	Do. Pl. rafter.
5/	6 3 — 8 — 4	7 0	Do. King-P.
5/	4 9 — 4 — 2	1 4	Ddt. Do.
5/2/	5 3 — 4 — 3	4 5	Add struts.
2/2/	30 6 — 6 — 4	20 4	Fir frd. in roof. Po. Pl. and Pur.
26/2	12 9 — 4 — 2	36 10	Do. Com. raft.
	30 6 — 7	17 6	Scarf 6 in. Sup. Do. 2-in. ridge (net.)
2/	30 6 — 4½ — 3	5 0	Fir fixed. Wall Plates.
		No. 10	Cleats for pur.
			Add ironwork.

The Abstract Sheet will be as follows :—

Fir fixed.	Fir framed in roof.	Fir framed in trusses.
ft. in.	*ft. in.*	*ft. in.*
5 8	20 4	26 11
	36 10	12 10
Cleats.	57 2	7 0
10		4 5
		51 2
		1 4
		49 10

The Bill will be as follows :—

CARPENTER.

ft. in.			*s. d.*	*£ s. d.*
6 0	Cub.	Fir in plates ...	2 10	0 17 0
57 0	”	” framed in roof...	3 4	9 10 0
50 0	”	” ” trusses	4 3	10 12 6
17 6	sup.	” ” 2-in. ridge (net) ...	0 7	0 10 3
30 0	run	” 2-in. rounded ridge roll birds-mouthed and spiked on ...	0 3	0 7 6
	No. 10	” in cleats to purlins	0 6	0 5 0
	No. 5	Hoisting trusses. ...	5 0	1 5 0
		Ironwork not specified.		
				£23 7 3

Example of Timber Piling.

Take the case of 20 vertical piles driven through 10 ft. of water 12 ft. into ground, 13-in. whole timbers rough hewn, shoes 28 lb. each, 8 of the piles to be cut off at a level of 6 ft. above the water, and the remainder carried up to a height of 30 ft. above water with one scarf each, scarfs 3 ft. long.

The Dimension Sheet will be as follows :—

		No. 20	Labour in driving 13-in. piles 12 ft. into solid ground through 10 ft. of water.
	ft. in.		12 ground. 10 water. 6 above. 1 head. — 29
8/	29 0 1 1 1 1	272 4	R. hewn whole timbers in piles 29 ft. long.
	ft. in.		12 ground. 10 water. 30 above. 3 scarf. 1 head. — 56
12/	56 0 1 1 1 1	788 8	Do. do. average over 27 ft. long, but not exceeding 35 ft.
		No. 20	Labour in pointing and shoeing 13-in. piles.
		No. 20	Labour in hooping 13-in. piles and use of hoops for driving.
		No. 8	Labour in cutting off heads of piles to uniform height.
		No. 12	Do. and waste in cutting off piles to break joint at 6-ft. intervals.
		No. 12	Labour in forming and bolting scarfs 3 ft. long in 13-in. piles.
		No. 20	Pile shoes 28 lb. each with cast points and wrot. straps, including spikes.

In this case the Bill may be written direct from the dimensions as follows :—

Provide all necessary plant, scaffolding, tackle, barge hire, etc.

PILE DRIVER.

		s. d.	*£ s. d.*
No. 20	Labour in driving 13-in. piles 12 ft. into solid ground through 10 ft. water.	20 0	20 0 0
	Carried to Summary		£20 0 0

CARPENTER.

ft. in.			*s. d.*	*£ s. d.*
273 0	Cub.	Memel fir rough hewn whole timbers in 13-in. piles each 29 ft. long.	2 6	34 2 6

ft. in.			s. d.	£ s. d.
788 0	Cub.	Memel fir, do., do., average over 27 ft. long but not exceeding 35 ft.	2 9	108 7 0
	No. 20	Labour in pointing and shoeing 13-in. piles ...	2 6	2 10 0
	No. 20	Do. in hooping or ringing 13-in. piles and use of hoops for driving	2 0	2 0 0
	No. 8	Do. in cutting off heads of 13-in. piles to uniform height	1 6	0 12 0
	No. 12	Do. do. in cutting off 13-in. piles to break joint at 6-ft. intervals...	5 0	3 0 0
	No. 12	Do. in forming and bolting scarfs 3 ft. long in 13-in. piles, including 3 bolt holes in each	7 6	4 10 0
		Carried to Summary		£155 1 6

SMITH AND FOUNDER.

cwt. qr. lb.		s. d.	£ s. d.
2 2 20	Wrot.-iron plates perforated for 1-in. bolts ...	20 0	2 13 7
1 1 19	In No. 36, 1-in. wrot.-iron screwed bolts and nuts, 14 in. wood measure	25 0	1 15 6
No. 20	In pile shoes 28 lb. each cast points and wrot. straps, including spikes ...	5 0	5 0 0
	(Bolts and Plates not specified.)		
	(If sheathed with copper at water level give detail.)		
	Carried to Summary		£9 9 1

SUMMARY.	£	s.	d.
Pile driver	20	0	0
Carpenter	155	1	6
Smith and Founder ...	9	9	1
Total	£184	10	7

Measuring Joiners' Work.

Example of 1½-in. square framed and flat 4-panel door 2 ft. 6 in. by 6 ft.

Dimension Sheet :—

	ft. in.		
	2 6 6 0	15 0	1½ dl. sq. frd. & flat 4-panel door & 3 oils b. s.
		No. 1	3-in. c.-i. butts.
		No. 1	7-in. iron rim lock and b.f. ft. in. 2 6 2/6 = 12 0 4/1¼ = 0 5 14 11
	14 11 0 6	7 5	1¼ dl. dble. reb. jamb ling. and bkg. & 3 oils. 14 11 4/4 = 1 4 16 3
2/	16 3 0 4	10 10	1-in. dl. wt. bd. & frd. grds. & 3 oils. 16 3 4/3 = 1 0 17 3
2/	17 3	34 6	1 × 3 architrave & 3 oils.

Abstract Sheet :—

JOINER AND IRONMONGER.

Sup.	Run.	Nos.
1½ dl. sq. frd. and flat 4-panel door.	1 × 3 architrave	Prs. 3-in.-c.-i. butts and screws.
15 0	34 6	No. 1
1¼ dl. dble. reb. jamb. lin. and backg.		7-in. iron rim lock and brass furn.
7 5		No. 1.
1-in. dl. wt. bd. and frd. grds.		
10 10.		

Fair Bill:

JOINER AND IRONMONGER.

ft. in.				£ s. d.
10 10	sup.	1-in. deal wrot., beaded and framed grounds.	-/6	0 5 5
7 5	„	1¼-in. do. double rebated jamb linings and backings.	-/10	0 6 2
15 0	„	1½-in. do. square framed and flat 4-panel door.	1/-	0 15 0
34 6	run.	1-in. × 3-in. do. architrave moulding.	-/3	0 8 8
	No. 1	pair 3-in. c.-i. butts and screws.	-/9	0 0 9
	No. 1	7 iron rim lock and plain brass furniture.	3/6.	0 3 6
		Carried to Summary	£	1 19 6

Measuring Windows.

Take now the case of three double-hung windows 3 ft. 9 in. by 4 ft. 3 in.; the taking off from the drawings will be as follows:—

Dimension Sheet:—

	ft. in.	ft. in.	
3/	3 9 4 3	47 10	Ddt. R.f. & s. & Ddt. distmp. & Add deal-cased frames 5 × 3 o.s. & w.s. & 1½ dl. mo. shs. dble. hg. wi. pat. sh. line. b.a.p. & i.w.
3/	3 9 1 0	11 3	Ddt. 3 oils to dado
3/	3 9	11 3	Ddt. 3 oils cut in bord.
3/	3 0	9 0	1⅓ × ⅛ galv. w.-i. tongue & bdg. in wt. ld. & gro. in oak & gro. in. Yrk. stn.
		No. 3	Hopkinson's pat. br. sh. fastnrs.
3/	4 0	12 0	1¼ in. × 2 dl. wrot. & reb. wind. nosg. & 4 oils. gr. & 2 v.
3/2/		No. 6	Notchd. ret. & mi. ends to do.
3/	3 9	11 3	gro. in oak. 3 9 4 3 4 3 4/3 = 1 0 13 3
3/	13 3	39 9	1 × 3 grds. frd. & splyd. & 4 oils. gr. & 2 v. 13 3 4/3 = 1 0 14 3
3/	14 3	42 9	3 in. archv. mo. & 4 oils. gr. & 2 v.
3/2/		No. 6	Mi. to do.
3/	2 9 3 7	29 7	26-oz. sht. gls. & glz. in 6 sqs.
2/3/		No. 6	Frames 4 oils.
		No. 3	Do. gr. & 2 v.
2/	1½ doz.	No. 3 doz.	Sqs. 4 oils.
		No. 1½ doz.	Do. gr. & 2 v.

The above is an extract from the actual dimensions of a small job. It will be seen that the quantity surveyor has to take note of all the surroundings, and not confine his attention merely to one trade; there is then less likelihood of any omission. The Abstract Sheet will be as follows for the joiners' work on above.

Abstract Sheet:—

Sashes and Frames.

4½ × 3 deal-cased frames, 5 × 2 oak sunk and wthd. sill and 1½-in. dl. mo. sashes dbl. hg. with br. a. p. and i. wts. and pat. sash line

47 10

Window finishings.

1 × 3 grounds frd. and splyd.
39 9

1¼ dl. wrt. and reb. window nosing.
12 0

3-in. ar. mo.
42 9

Mi.
6

notchd. retd. and mitrd. ends
6

Groove in oak
9 0

The same items transferred to the Fair Bill will be as follows :—

SASHES AND FRAMES.

ft.	*in.*				£	*s.*	*d.*
48	0	Sup.	4½ × 3 deal cased frames, 5 × 2 oak sunk and weathered sills and 1½ deal moulded sashes, double hung with patent lines, brass axle pullies and iron weights	1/4	3	4	0

WINDOW FINISHINGS.

ft.	*in.*						
12	0	run	1¼ deal wrot. one side and rebated window nosing...	-/3	0	3	0
			No. 6 notched returned and mitered ends to do.	-/3	0	1	6
42	9	,,	3-in. architrave moulding ...	-/3	0	10	8
			No. 6 mitres ...	-/3	0	1	6
9	0	,,	Groove in oak ...	-/1½	0	1	2
					£4	1	10

To enable this method of taking off quantities to be fully understood the subject requires special study from any of the well-known text-books, such as Leaning's "Quantity Surveying" (Spon; 25s.), and Banister Fletcher's "Quantities: a Guide to the Best Methods Adopted in the Measurement of Builders' Work" (Batsford; 6s net).

ROOF COVERINGS AND ROOF GLAZING.

Roofing Materials.

Asphalt.—Description: A compound of bitumen and calcareous matter used in forming flat roofs for drying grounds, etc., in towns, and might be much more used. Generally laid on thin concrete. Advantages: Waterproof, fireproof, and easily applied to flat roofs. Has no joints for entrance of moisture. Disadvantages: Very heavy, requiring a strong flooring to support it. Softens under the direct rays of strong sun. Fig. 1004 is a section of a flat asphalt roof carried on timber joists ceiled below; provision for gutters, fall, outlets, etc., is shown; and the general level may, if desired, be kept up ¾ in. to allow of a shallow gutter at the lower end, the bottom of the gutter being level with the outlet.

Concrete.—Description: An artificial compound of lime or cement mixed with sand, water, and some hard material for the aggregate, as broken brick or stone. Used for roofs, on ceiling joists, or in flat arches, floated on top with cement mortar or a layer of ½ in. to ¾ in. of asphalt. Advantages: Fireproof and durable. Disadvantages: Very heavy; requires great care in filleting the angles to brickwork, etc. Concrete and asphalt are used on flats, and are very suitable where access is required on to a roof, but they require heavy timbers. It is not usual to construct flat roofs in the way indicated in Fig. 1005, but the writer does not see any objection to a trial being made if special circumstances seem to render it desirable. Such a system may be suitable for roofs of back bedrooms and offices of cottages; the joists are covered with tongued 1-in. boards, then a layer of felt, and finally 2½ in. of cement concrete. The exposed woodwork is wrought. The span is not given, but the joists would not be less than would be used for a floor of the same span. The cement concrete should be rather rich—say not more than 4 of ballast to 1 of cement—and the surface should be floated with neat cement. Special care will have to be taken at the edges in order to prevent damp from reaching the felt and wood, a strip of lead being inserted as shown in Fig. 1005. The cement should be well matured to prevent shrinking and cracking, but must be perfectly sound and fresh, and the concrete must be thoroughly mixed both before and after the addition of water.

Copper.—Description: Used principally for covering wood spires and domes, in sheets weighing about 16 oz. per ft. super., or thicker. Copper is a good covering for steeples and steep roofs, where the expense can be allowed. It gets a black colour very soon in ordinary air from oxidation, etc. Slope, say, 4° to 75°, weight 80 lb. to 120 lb. per square. Advantages: Is very light, fireproof, and permanent. Although it rapidly oxidises, turning black, the oxidation is not deep, and tends to preserve the remainder. Disadvantages: Is very expensive, and conducts heat readily.

Corrugated Iron (galvanised) is useful for temporary buildings and for covering sheds cheaply. May be laid at any angle. Weight about 350 lb. per square (16 B.W.G.). Decays rapidly in town air, unless painted every three years.

Glass of the kind used for roofing varies from ordinary window glass fixed with putty to large sheets of rolled glass ¼ in. or more in thickness. The advantages are the full admission of light and ease of cleaning—the disadvantages are the great weight and the non-exclusion of the sun's heat.

Lead.—Description: Used for covering flats and occasionally domes. Principally for portions of roofs, where it is better adapted than any other material for covering joints to prevent access of moisture. Lead is suitable for covering very flat roofs, and also small portions of ordinary roofs where it is required to keep out the weather. On steep roofs it is unsuitable, as the alternate expansion and contraction cause it to creep down the slope. Slope, say 4°; weight, 550 lb. to 850 lb. per square. Advantages: Easily dressed over irregular parts and soldered to keep out wet. Is very durable when exposed to the weather, much more so than zinc. Disadvantages: Not adapted for pitched roofs, as it creeps down by alteration of temperature. Requires nailing all over if on a slope. Is more costly and much heavier than zinc. Must be fixed with copper or composition nails, as iron would rust away and leave perforations for entrance of moisture.

Shingles.—Description: Are slabs of split oak. Formerly much used, and still to be seen on old stone buildings in country districts. Small slabs of similar material are occasionally used in Gothic work at the present day. Advantages: Very light and easily fixed. Very picturesque for covering church spires and high-pitched roofs. Disadvantages: Require a steep pitch to throw off the water quickly. Very inflammable.

Slates and **Tiles** are discussed on pp. 262–274.

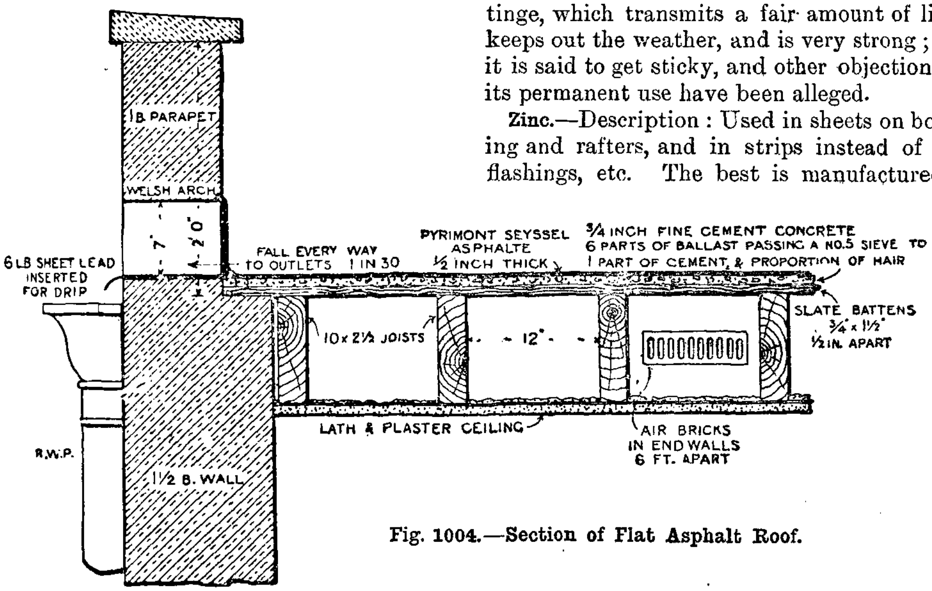

1 B. PARAPET
WELSH ARCH
7' 0"
6 LB SHEET LEAD INSERTED FOR DRIP
FALL EVERY WAY TO OUTLETS 1 IN 30
PYRIMONT SEYSSEL ASPHALTE ½ INCH THICK
¾ INCH FINE CEMENT CONCRETE 6 PARTS OF BALLAST PASSING A NO.5 SIEVE TO 1 PART OF CEMENT, & PROPORTION OF HAIR
SLATE BATTENS ¾" x 1½" ½ IN. APART
10 x 2½ JOISTS
12"
LATH & PLASTER CEILING
AIR BRICKS IN END WALLS 6 FT. APART
R.W.P.
1½ B. WALL

Fig. 1004.—Section of Flat Asphalt Roof.

Tarred Felt is very useful on rough sheds and temporary buildings. It is light and cheap, but requires tarring and sanding every two years, and renewal in seven years.

Willesden Paper is cheap, and suitable for temporary work. It is a semi-transparent green waterproof paper.

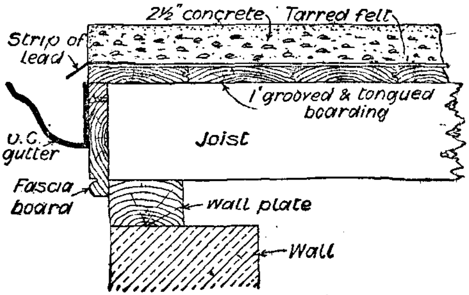

2½" concrete
Tarred felt
Strip of lead
1' grooved & tongued boarding
Joist
U.C. gutter
Fascia board
wall plate
Wall

Fig. 1005.—Section of Flat Concrete Roof.

Wire-wove Roofing is wire netting or gauze coated with some form of gelatine of a green tinge, which transmits a fair amount of light, keeps out the weather, and is very strong; but it is said to get sticky, and other objections to its permanent use have been alleged.

Zinc.—Description: Used in sheets on boarding and rafters, and in strips instead of lead flashings, etc. The best is manufactured at the Vieille Montagne Works in Belgium. Zinc is very light compared with any of the foregoing, and its use saves material in roof timbers. It is durable in a pure atmosphere, where it gets a thin coating of oxide or carbonate which protects the remainder. In the atmosphere of towns, especially where acid fumes are in the air, it decays very rapidly. Slope, say 4°; weight, 150 lb. per square. Advantages: Can be laid to a very flat pitch with only slope enough to drain off the water or at any angle as required. Very light, and hence can be laid on roof timbers of small scantling. Is tough and strong, and can be used in exposed situations. Perfectly non-absorptive of moisture. Cheap. Disadvantages: The colour and lustre are objectionable from an architectural point of view, except perhaps on a pier pavilion. Zinc expands and contracts with change of temperature more than any other metal. Transmits heat readily. Is rapidly destroyed by chemical action from the acids in the rainwater of towns, and by galvanic action if in contact with iron, copper, or lead. Takes fire and blazes furiously if sufficiently heated.

Slates.

The advantages of slates are that they are cheap, and can be obtained in great variety of quality, size, and thickness. The colour tones well with either brickwork or masonry. The surfaces being smooth, the slates lie close together, and being thin and regular they look very neat, and are especially suitable for Classic work. They have less weight than tiles, and hence require lighter roof timbers. They are very non-absorbent when of good quality. They are easily replaced when broken, and therefore economical. Slates are moderate in cost, extremely durable, bad conductors of heat, moderate in weight, and have a neat appearance. Their disadvantages are that they are liable to be lifted and stripped off by the wind in exposed places, especially if head nailed or laid flatter than 26½° or 30°. They are somewhat brittle and liable to be damaged when clearing out gutters, or repairing chimneys, etc.

Qualities of Good Slates.

Good slates are hard and tough to resist breaking, and give a good ring when struck with the knuckles. Flat but somewhat rough surfaces, close fine grain, and firm edges. Not fracturing when holed or squared. Uniform in size, colour, and thickness, without light-coloured veins, and free from patches of "marcasite," or white iron pyrites, which cause decay on exposure. Non-absorbent; the most absorbent generally feel smooth and greasy.

Slate Formation.

Slates are thin rectangular slabs of an argillaceous rock, compact and fine-grained, with distinct cleavage planes. It must be understood that the cleavage planes in slate are not the same as planes of stratification in sandstones. A true slate is an argillaceous rock that has been transformed into a semi-crystalline mass by heat and pressure, the original planes of deposit being lost and new cleavage planes formed. Fig. 1006 shows a section through a mass of slate in situ. The convolutions or waves are produced by the compression of the original line of deposit due to the cooling and contraction of the earth's crust; the denudation of the upper part by the weather produces the surface of the ground, and the parallel lines show the planes of cleavage. It will be observed that these planes are more or less inclined, and in some places are perpendicular to the original beds, and could hardly be parallel with them except on a very short fold. Cambrian slates (for example, Penrhyn) and Lower Silurian (for example, Llandilo) are the same in this respect. It has been suggested that the aggregations of atoms of the slate were originally deposited circular, in layers as shown in Fig. 1007, that the weight of superincumbent deposits compressed these atoms as shown in Fig. 1008, making horizontal cleavage, and that the compression due to the cooling of the earth's crust caused the atoms to take a more or less vertical position, as shown in Fig. 1009, with corresponding vertical cleavage. The Stonesfield slates from the Oolitic limestone formation near Stamford (Lincolnshire) and elsewhere are not true slates, but merely thin slabs of stone that split easily into thin layers along the planes of bedding.

Varieties of Slates.

Roofing-slates are obtained from various quarries in the *north* and *west portions* of the United Kingdom, occurring chiefly in the geological formations known as Cambrian and Silurian.

English Slates.—The chief quarries for these are at Kendal, in Westmorland, where thick, coarse, hard, but tough and durable slates of a greenish colour are obtained. Westmorland slates now come largely from Ambleside, Langdale, and Thang Crag at Windermere. Being more siliceous than Welsh slates, they do not cleave so well, but are fully as durable. They have been used on several large buildings in London. The Delabole Quarries, in Cornwall, supply slate slabs of good quality, and small quantities are obtained from other parts of England.

Welsh Slates.—These come principally from the Bangor, Penrhyn, Dinorwic, and Portmadoc quarries, in Carnarvonshire. They are smooth slates of a bluish or purplish colour, used for buildings of all classes. There are also good slates obtained from the Festiniog district of Merionethshire ; these are blue slates, with a very good natural cleavage, splitting finer and to a more uniform thickness than others. Many minor quarries in other districts are also worked.

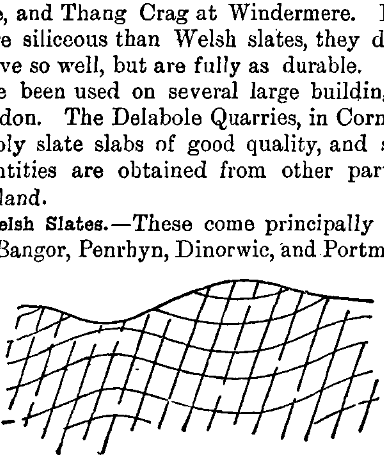


Fig. 1006.—Section through Mass of Slate in situ.

Scotch Slates.—The principal Scotch quarries are in Perthshire and Aberdeenshire. They supply thick and coarse slates of varying colour but generally of a bluish grey, with a large proportion of iron pyrites. There are also quarries of dark-blue slates at Ballachulish, in Argyllshire. As a rule, the darker slates are softer and absorb more moisture, and are therefore more liable to decay.

Irish Slates.—These are obtained from Killaloe. They are of a dull bluish-grey colour, very durable, but thicker and heavier than Welsh slates.

Foreign Slates.—American, Canadian, and French slates are occasionally met with in the market, but are not considered to be so good as British slates.

Defects in Slates.

The principal causes of decay in slates are damp and frost affecting the common varieties, and the decomposition of marcasite or white pyrites when contained in them. The selected slates should be hard and toughish, free from veins and patches, and uniform in colour. The effect of iron pyrites upon slates depends upon the character of the pyrites. The presence of the yellow, brassy-looking pyrites, although a blemish, does not much affect the endurance of the slate. White pyrites, or marcasite, generally of a dull colour, without lustre, is very objectionable, as it rapidly decomposes by exposure to the atmosphere, and no slates containing it should be used for roofing. A pale greenish rounded patch on slates, something like a drop of candle grease that has spread and cooled, about 1½ in. by ½ in., is chlorite or some allied substance, and is unsightly but not particularly detrimental.

Testing Slates.

To judge of the quality of a slate, stand it up to half its depth in water for twelve hours, when no sign of moisture should appear more than ¼ in. above the water-line. The slate having been thoroughly dried and then immersed in water for twelve hours, the difference of weight before and after immersion should not exceed 1 per cent. A good slate, when breathed upon, should not emit a strong clayey odour ; if it does, it will probably not weather well (see also p. 132).

Sizes of Slates.

The principal market sizes of slates are known as doubles, 13 × 6 ; ladies, 16 × 8 ; countesses, 20 × 10 ; duchesses, 24 × 12 ; and there are others larger and smaller, but not in common use. Two sizes most in use are duchess and countess, each $\frac{3}{16}$ in. thick for first quality, ¼ in. thick for second quality.

Fig. 1007.

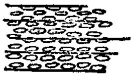


Fig. 1008.

Fig. 1009.

Figs. 1007 to 1009.—Diagrams illustrating Slate Formation.

Weight of Slates.

Slates vary in weight, of course, according to size, the larger slates being thicker and more suitable for very flat roofs. Large slates are for use on, say, a slope of 22°, their weight per square being 900 lb. to 1.100 lb. Ordinary slates are for, say, a slope of 26½°, their weight per square being 550 lb. to 800 lb. Small slates are for, say, a slope of 30°, their weight per square being 450 lb. to 650 lb. Thus, as shown, large slates may be laid to a flatter slope, but what would be gained in reduction of height of trusses would be lost in the greater stresses due to reduction of depth. Large slates are also heavier, requiring greater scantlings of timber for their support. First quality empress slates, 26 in. by 15 in., weigh 80 cwt. per M (which is the symbol for a "long thousand") of 1,200, and cover 1,445 sq. ft. laid with a 3-in. lap. Countesses, 20 in. by 10 in., weigh 40 cwt., and cover 708 sq. ft., with the same lap. Therefore, putting aside other considerations and taking only the surface covered, empresses at £14 per M will be equal to $\frac{100 \times 14 \times 20}{1445}$, say 19s. 4½d. per square, and countesses at £6 5s. per M will be equal to $\frac{100 \times 125}{708}$, say 17s. 7½d. per square. In calculating the weight of slating on a roof, proceed as in the following example: Assume that the slates measure 24 in. by 12 in.; that an average slate weighs 7 lb.; that the slates are laid with a lap of 4 in., and that it is required to find the weight of slates which cover an average square of roof. The length of slate occupies two margins and a lap; the margin or gauge is therefore $\frac{24-4}{2} =$ 10 in. The margin covers two slates throughout, and one more for the width of lap only, making an equivalent to one whole slate for each margin, or $\frac{10}{12}$ of a foot. If $\frac{10}{12}$ of a foot requires 1 slate = 7 lb., 100 sq. ft. will require $\frac{7 \times 100 \times 12}{10} =$ 840 lb. of slating.

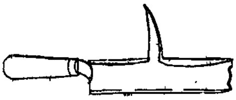


Fig. 1010.

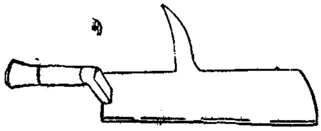


Fig. 1011.

Figs. 1010 and 1011.—Slater's Zax or Axe.

Cutting and Holing Slates.

A slate is cut by resting it on the "dog" (an iron straightedge driven into a wooden stool by its two points) and striking it with the zax, or by making a row of holes with the pick on the back of the zax and breaking it across the holes. The zax, sax, slater's axe, trimmer, dressing knife, or hewing knife, with pick for holing the slates to receive the nails, is shown in two forms by Figs. 1010 and 1011. The pick is bent in order to lie concentric with the arc through which the centre of gravity of the axe moves when in use, and is sufficiently near the handle to be at the centre of percussion to prevent any jarring of the slater's hand.

The Slater's Hammer.

The slater's hammer is clearly shown by Fig. 1012. In this illustration A indicates the point for holing slates, B the head for driving nails, and C the claw for pulling out defective nails.

Slaters' Nails.

Slaters use nails of cast-iron, malleable iron, zinc, copper, and composition (see Fig. 1013). Composition nails, if made from a suitable alloy, are best, being hard and not liable to corrosion; they are cast from an alloy of about 7 copper to 4 zinc. When made of a really good strong alloy they are the best for superior work. Ship nails are very similar; these are composed of copper 10 parts, zinc 8 parts, cast

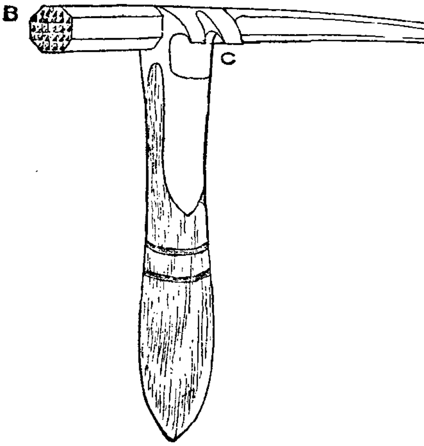


Fig. 1012.—Slater's Hammer.

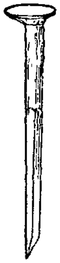


Fig. 1013.—Slater's Nail.

iron 1 part, but probably the latter is often omitted. Any brass founder could supply the composition nails wholesale at about 9d. per lb. or £4 per cwt., but the composition ought to be guaranteed. Wire nails when used for slating are very subject to rust; they might be galvanised, if the cost is not deterrent; but at least, as a slight protection, the nails might be dipped in boiled oil and dried before use. Composition nails, however, are generally considered to combine the happy medium of cheapness with efficiency.

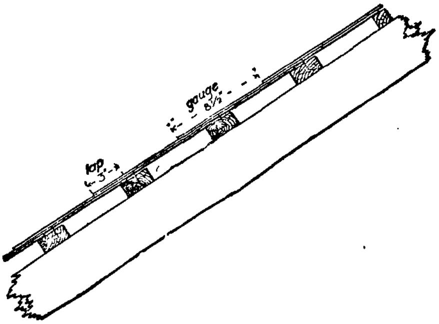


Fig. 1014.—Countess Slating Centre-Nailed.

How Slates are Laid.

Slates are laid on battens or on close boarding; they are centre-nailed, or nailed at the head, and they are laid with closed or open bond. The cheapest mode is open bond head-nailed on battens, but this is only fit for outhouses and tool-sheds. When laid upon boarding a layer of asphalted roofing felt or hair mortar may be interposed with advantage, to prevent draughts and extremes of temperature; and if slates be nailed to battens over the felt a small circulation of air is permitted, which helps to preserve the felt from decay. Centre-nailing is preferable to head-nailing in exposed positions, as the wind has then only half the leverage to loosen the slates. Fig. 1014 shows countess slating centre-nailed; Fig. 1015 shows the same head-nailed. When head-nailed, the lap is measured from the nail-hole, reducing the gauge by ½ in. When felt is not used the slates may be "shouldered"—that is, the heads for a width of 2 in. are embedded in hair mortar, coloured with coal ashes, to keep the slates down tight at the tails. This is particularly suitable for exposed situations or rough slates. When laid on battens the slates are frequently rendered all over the underside with lime and hair; but sometimes they are only "torched," or pointed on the inside, which is not so effective or durable. Tails of slates may be secured in specially exposed positions by patent clips entering the joint between the slates in the course below and turned over the tail of the slate above, as shown. In all slating a double course is laid at the eaves, where the work of laying is begun (see Fig. 1016). This merely consists in cutting off the margin from one row of slates, but in common work the slates are laid lengthwise instead of being cut. Two nails to each slate are inserted, 1½ in. from the sides. The slates have a gauge or margin from 6 in. to 10 in., and a lap of 2½ in. to 3½ in., varying with size of slate and mode of

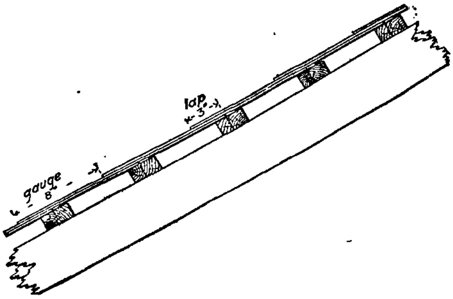


Fig. 1015.—Countess Slating Head-Nailed.

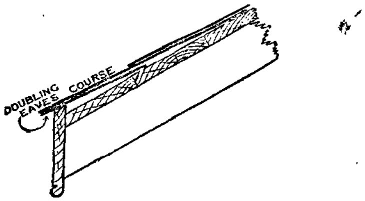


Fig. 1016.—Double Eaves Course of Slating.

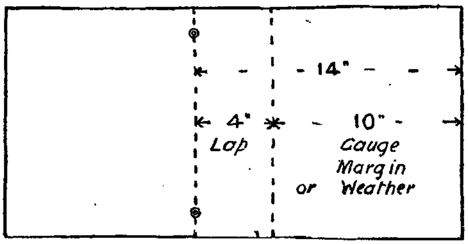


Fig. 1017.—Duchess Slate Dressed and Holed.

nailing. A duchess slate (24 in. by 12 in.), dressed and holed with a lap of 4 in., is shown by Fig. 1017. If the slate is head-nailed the margin or gauge will be 9½ in. and the distance of the nail holes from the tail will be 23 in. If centre-nailed as shown, the margin will be 10 in., and the distance 14 in.

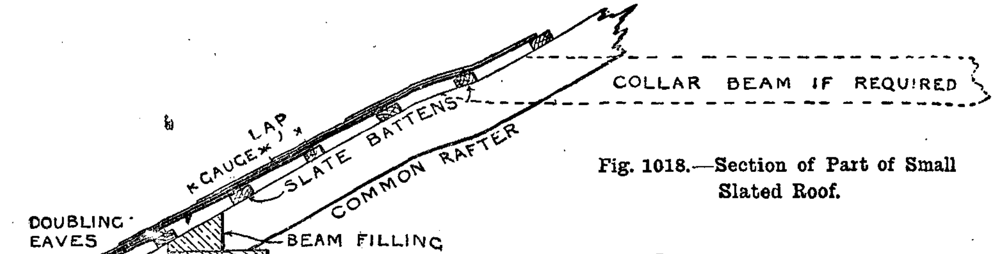


Fig. 1018.—Section of Part of Small Slated Roof.

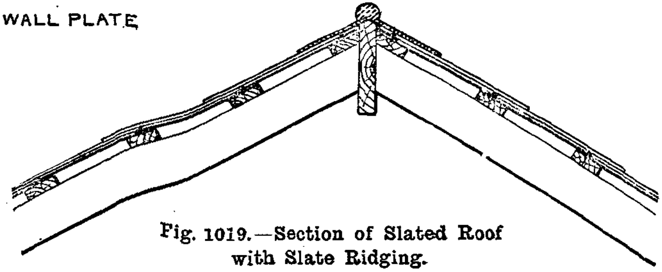


Fig. 1019.—Section of Slated Roof with Slate Ridging.

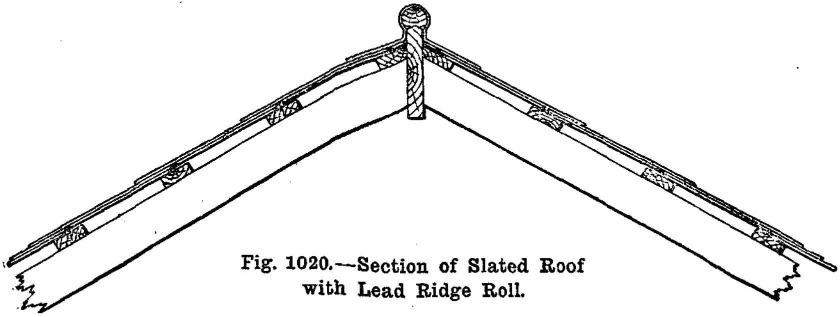


Fig. 1020.—Section of Slated Roof with Lead Ridge Roll.

Procedure in Slating.

In a new house the slating is usually done before the bricklayers' scaffolding is removed. If old-fashioned cripples (brackets) are to be used these will be fixed at a convenient height, say 3 ft. below eaves. Cripples are used in pairs for carrying a temporary scaffold of one or two boards, and are fixed against the wall with wall hooks and wedges. Better arrangements have taken their place. The roof is assumed to be complete with rafters, valley boards, etc., fixed, and plumbers' work ready. The slates are then dressed and punched with the zax already shown. The battens should be 3 in. by 1 in., nailed to the rafters with 2½-in. cut nails to suit the gauge of the slates, say 10 in. centre to centre, starting with the tilting fillet. The under-eaves or doubling course of slates is laid first in the same manner as the others, but with the gauge or margin portion cut off. It is improper to lay them lengthwise, as is sometimes done in common work to save waste. The slates should be fixed with 2-in. galvanised iron or composition nails, course by course, with the side joints truly in line. "Keeping the perpends" is important; this means keeping the vertical joints of alternate courses in straight lines from eaves to ridge. At the hips and valleys the slates will have to be cut to the proper angle, for which a template is made to which the slates are dressed. Creeping boards, or "creepies," are used on the lower slates to reach the higher portions. At the ridge a

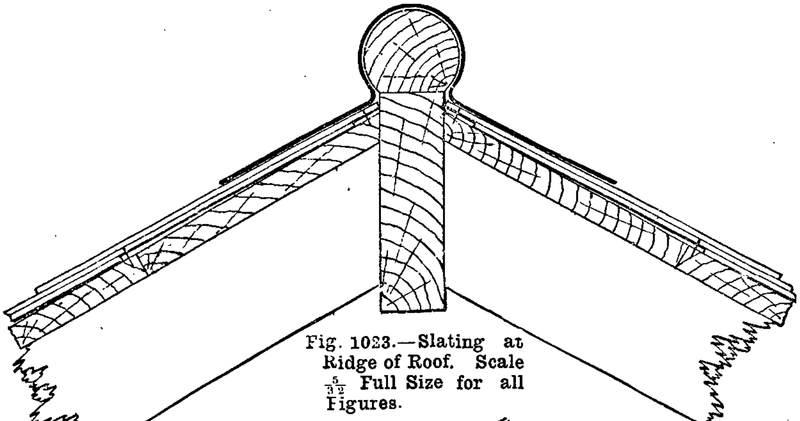


Fig. 1023.—Slating at Ridge of Roof. Scale $\frac{5}{32}$ Full Size for all Figures.

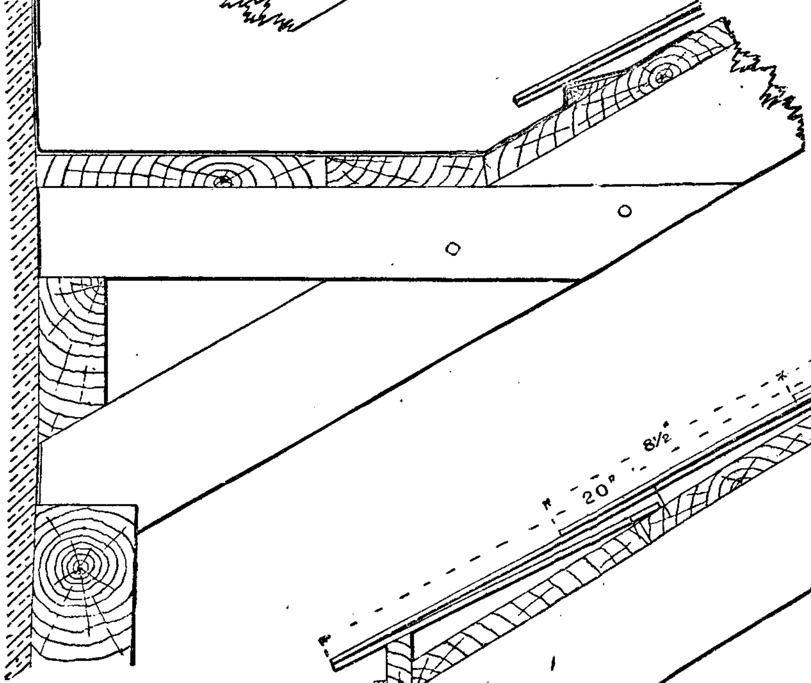


Fig. 1022. — Slating at Eaves with Parapet Gutter.

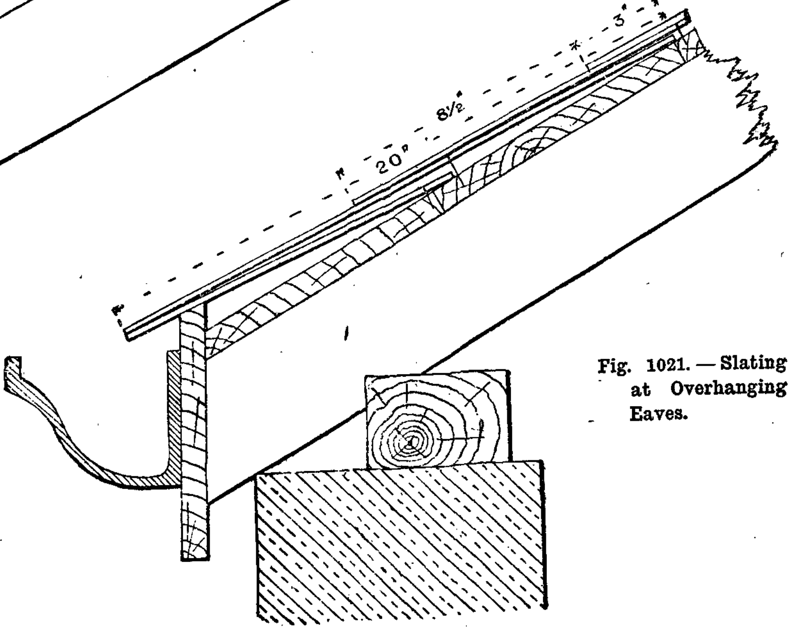


Fig. 1021. — Slating at Overhanging Eaves.

double course should be again used to protect the joints. Waste occurs at the eaves and ridge, hips and valleys, chimney stacks and dormers, skylights, traps, gable ends, etc.

Torching Slates or Tiles.

Torching, already alluded to, is the pointing of the joints of slates or tiles on the underside when laid on battens, to prevent cold winds and dust from blowing through. The material used is coarse stuff—that is, lime and hair mortar, with which the whole of the underside is sometimes rendered to keep the space between roof and ceiling cool. When laid on close boarding, the slates or tiles are sometimes bedded in mortar as an equivalent for the torching, which cannot then be done. Tarred or so-called asphalted roofing felt is used for similar purposes on either battens or boarding.

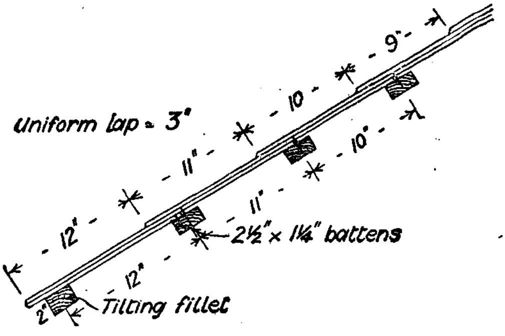


Fig. 1024.—Graduated Courses in Slating.

Construction of Slate Roofs.

The section of part of a small span roof is presented by Fig. 1018. The countess slates have a 4-in. lap, and are centre-nailed on 2-in. by ¾-in. battens. There is a 3½-in. half-round eaves gutter, and a 1-in. beaded fascia board. The rafter projects 9 in. from face of wall. Pitch of roof is 30°. Fig. 1019 is a cross-section through the ridge of a slated roof showing three courses of duchess slates centre-nailed on 3-in. by 1-in. battens and a 9-in. by 1½-in. ridge-piece finished with slate ridging. Fig. 1020 is a similar section showing the details of a lead ridge roll. The ridge and eaves of a roof laid with countess slates to a 3-in. lap are shown by Figs. 1021 to 1023. A section through overhanging eaves is presented by Fig. 1021, and a section through eaves gutter behind a parapet wall by Fig. 1022, whilst Fig. 1023 shows a section through the ridge.

Graduated Courses in Slating.

When graduated courses are worked in slating, the lap remains uniform, but the gauge varies with the length of the slate, so that the slates have to be sorted out to uniform length for each course, and the battens put the same distance centre to centre as the gauge of slate. Suppose, for instance, the first four courses have the gauge 12, 11, 10, and 9 in. respectively, the battens will be fixed as shown in Fig. 1024.

Eaves Fillet for Roof.

The extra thickness of an eaves fillet or tilting fillet is to make up the difference between the two thicknesses of slate at that point and the three thicknesses at the other fillets or battens; the fillet should therefore be at least ¼ in. thicker. The fillet also helps to keep the course joints closer. In tile roofs the eaves fillet is often made much thicker than in slate roofs, in order to throw up the eaves course for effect. This is carried farther in some cases by making a bell-cast with sprockets on the feet of the rafters or spars.

Laying Westmorland Slates.

Westmorland slates may be laid in single courses direct upon the rafters and without battens, but if laid on battens like ordinary slating the spacing of the battens will be found thus: $\frac{1}{2}$ (length of slate − lap) = gauge of battens for centre-nailing. Westmorland slates may be laid upon battens or boarding with the usual lap and gauge and in the usual manner, but when of large size and considerable thickness may be economically laid in the manner described below. "The rafters are placed at a clear distance apart, about 1½ in. less than the width of the slates. Down the centre of each rafter is nailed a fillet, thus forming a rebate on each side, in which the edges of the slates rest, being secured by black putty, or (as this looks smeary and uneven) by a second fillet 2 in. wider than the first fillet and nailed over it so as to cover the edges of the slates and hold them down. Each slate laps about 3 in. over the slate below; only half the number of slates is required in this method as compared with the ordinary method of slating, and no boarding or battens are necessary." If the slates vary in length, the longest should be used at the eaves. If the slates vary in width, the same width must be used in vertical lines up the slope of the roof, but horizontally the slates may be used alternately long and short. The following explains one method of setting up the laths and holing the slates for a grey Westmorland roof : Sort out the slates into graduated lengths, then suppose them to be head-nailed and to have a lap of 3 in. ; the margin for the first course and the position of the batten or lath will be found as follows :

	in.
Length of slates in first course, say ...	26
Length of slates in second course, say ...	25
Difference =	1
Net lap required	3
Nail hole to head of slate	1

Length of slates in first course 26 in. less 5 in. = 21 in.

	in.
21 divided by 2	10½
Add difference of length	1
Gives margin of first course	11½
Add net lap	3
Gives centre of first row of battens ...	14½

The battens for the second course will be 14½ in., less difference in length of slates 1 in. = 13½ in. from the last batten, centre to centre, and the spacing for the remaining battens will be found in a similar manner.

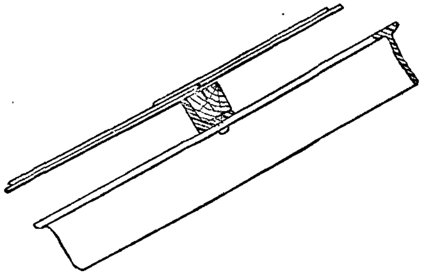


Fig. 1025.—**Slates Centre-Nailed to Angle-Iron Purlins on Steel Roof.**

Fixing Slates on Steel Roof.

Retort-houses are sometimes covered with slates centre-nailed to 2-in. by 2-in. square battens resting in light angle-iron purlins placed to suit the gauge of the slating, as Fig. 1025. It would no doubt be possible to fix the slates to the purlins by short lengths of iron wire direct, as *Fig. 1026*, but the writer has never seen it done.

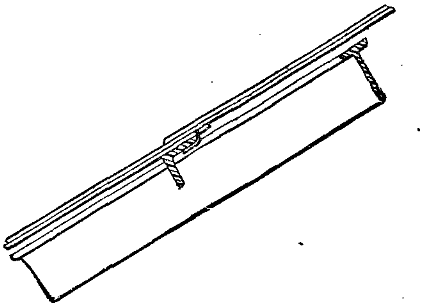


Fig. 1026.—**Slates fixed with Iron Wire to Steel Roof.**

Determining Number of Slates Required for Roof.

The necessary formula for ascertaining the number of slates required for a roof is given below. Length of slate = twice the gauge + lap. Gauge = $\frac{\text{length slate} - \text{lap}}{2}$. Net area covered by one slate = gauge × width. Distance of holes from tail end, centre-nailing = gauge + lap. Slates required per square = $\frac{100 \times 144}{\text{gauge} \times \text{width}}$. Thus with 18-in. by 10-in slates (lap, 3 in.), gauge = 7½ in., and number of slates required per square = $\frac{100 \times 144}{7{\cdot}5 \times 10} = 192{\cdot}8$, say 200.

Determining Number of Squares and Number of Slates.—The number of squares of 100 ft. sup. should be taken from the bill of quantities or determined as follows : Take from the section the distance up the slope of the roof and multiply this distance by the length along two sides, the quantity for a hipped or gabled roof being the same. Deduct the space occupied by chimney, skylights, traps, dormers, etc., and add the length by 6 in. for waste in cutting around all deductions. Add length by 6 in. on each side for waste in cutting to hips, valleys, and irregular gables. No allowance for square gables or ridge. Allow length by gauge or margin for waste on doubling course at eaves and curb. Total this up to arrive at the amount inserted in the bill of quantities. Then according to the size of slate and margin shown will be the covering power of each. For example, countess slates 20 in. by 10 in., centre-nailed, with 3-in. lap, the gauge will be 8½ in. and the exposed surface 10 × 8½ = 85 sq. in., and if the roof contains 8 squares = 800 sq. ft., the number of slates will be

$$800 \div \frac{85}{144} = \frac{800 \times 144}{85} = 1356.$$

Measuring Slaters' Work.

In measuring slaters' work the length of the roof is taken from the elevation or the plan, and from the section for the width up the slope, multiplied together for area, and by 2 for the two sides. Whether the roof has gabled ends or hipped ends, the total area is the same. The kind of slating must be described, the gauge or the lap, and the kind and the number of nails to each slate. Chimneys, skylights, dormers, etc., are either considered as equal in saving slates and causing extra labour, and are therefore not accounted for; or by the better method the space the chimneys, etc., occupy is deducted from the slating, and an allowance of 6 in. all round is made in order to compensate for extra labour in cutting, and waste. Also at the edges of the curbs and eaves an allowance is made of the length by the gauge. Another allowance is made of 6 in. on each side for cutting to hips, valleys, and rakes. No allowance is made for cutting slates at the ridge or at a gable end. When the total is found it is divided by 100 in order to bring it to squares, and under this denomination is billed. Slating on conical roofs should be kept separate and fully described; the allowances would be the same as above. The writer cannot say whether any different allowances are made in particular districts, but this is the usual method.

Specification for Slater's Work on a First Class Dwelling House.

The main roofs to be covered with best quality Westmorland green slating to approved sample laid in gradually diminishing courses from eaves upwards, 15-in. gauge at eaves and 10-in. at ridge, to a 3-in. lap, and with extra stout copper nails 2 in. and 1¾ in. long, 90 and 110 to the lb. respectively. The other roofs to be covered with best Bangor countess slating, centre-nailed and laid to a 3-in. lap, with two strong copper nails to each. The perpends of all slating to be kept true from eaves to ridge. Double course to be put to all eaves, each slate to be of full width and secured by two copper nails. The slating to be accurately cut to fit all hips, valleys, skylights, chimney stacks, and other deductions. Each slate to be hung with two copper nails, and no piece less than 6 in. wide.

Specification for Slater's Work on Roof in Very Exposed Position.

The roof to be covered with Engert & Rolfe's best inodorous asphalted roofing felt, laid with 2-in. lap upon 1-in. rough boarding, edges shot and closely laid. Slate battens 2 in. by ¾ in. to be laid over the felt to 8-in. gauge to suit countess slating with thicker batten as tilting fillet at eaves. The slates to be best Penrhyn or Portmadoc slates of countess size, centre-nailed with two copper nails to each, and laid to a 4-in. lap, cut and trimmed wherever necessary. All the slates to be "shouldered" or bedded for about 2 in. at their heads in hair mortar mixed with coal ashes. To be laid in straight lines, vertical and horizontal, the eaves and ridge courses to be double, the eaves projecting 3 in. The ridges and hips to be covered with 4-lb. lead in wide pieces, nailed down at the edges under the slates, and dressed to the rolls after slates are laid, forming a double thickness on the surface of the slates without nail holes. The valleys to be laid with 5-lb. lead and the gutters with 7-lb. lead over proper layer boards and fillets, turned up 6 in. under the slates, and 4 in. against the wall with apron flashing. All necessary drips, cesspools, and outgoes to be formed in a workmanlike manner with 7-lb. lead bossed and soldered. Leave all perfect on completion.

Tiles.—Description : Made of clay pressed in moulds of various shapes and burnt in a kiln. The best are "best pressed Broseley tiles with projecting nibs to fit over laths and laid to 3½-in. gauge on 1½-in. by 1-in. sawn fir laths, and secured with two stout composition nails and torched on underside." The commonest are pantiles, only used for covering sheds, very cheap, laid to a slope of 24°, weight 1,200 lb. per square; require heavy roof timbers, and hold moisture a long time, therefore not suitable for ceiled roofs. Plain tiles are smaller, but being laid with more lap are heavier, say 1,800 lb. per square ; suitable for slope of 30° to 60°. Principal cause of decay is chipping and weathering by frost. Should be burnt very hard. Advantages : Picturesque and very suitable for Gothic work. May be had in various shapes of red, purple, and blue-black. They are non-conductive of heat. Disadvantages : Heavy, and therefore require stout roof timbers. Most kinds absorb much moisture, increasing the weight, and communicating the moisture to the rafters, causing them to rot. Require a high-pitched roof to keep them from stripping by the wind or permitting rain to drive under.

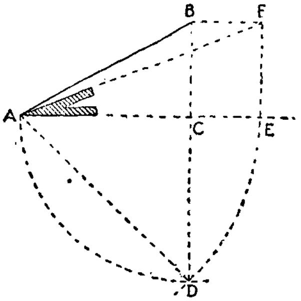


Fig. 1027.—Obtaining Pitch of Hip Tiles.

How Tiles are Laid.

Plain tiles are small rectangular slabs of burnt clay, generally about 10 in. long, 6 in. wide, and about ½ in. thick. The tiles are laid on laths of fir nailed to the rafters, being hung from the laths by oak pegs driven through holes near the upper edge of the tiles. Sometimes, instead of using pegs, little projecting nibs are formed on the upper edge of the backs of the tiles, by which they are hung on to the laths. The arrangement of the tiles is similar to that employed for slates ; the tail of each tile rests upon the tile below for a length of about 6 in., the gauge being 4 in. and the lap over the head of the tile next but one below about 2 in. The lap in tiling is commonly understood to be the projection of a tile over the next but one below it ; but some authorities hold that the lap, both in tiling and slating, should be measured from the nail or peg holes in the next tile but one below. In "Notes on Building Construction," slating is said to be measured from the nail hole, while the plain tiling is given as "about 2-in. lap," and is measured from the head of the tile. In exposed places each tile is bedded upon the one below it in hydraulic mortar or cement. Tiles require heavy roof timbers, as they weigh more than twice as much as slating —say 1,800 lb. per square against 700 lb. The larger tiles, called pantiles, are only used for common work; although larger, pantiles weigh, when fixed, only two-thirds the amount of plain tiling, owing to the smaller number of laps.

Laying Yorkshire Slates or Tile Stones.

Yorkshire slates, or grey slating, is the name given to the thin flags or tile stones, or stone slates, largely used for roof coverings in the northern counties. They are generally from ½ in. to 1 in. thick, and hung to the battens by wooden pegs about 3 in. long, and laid close. Owing to their weight they require heavier rafters or spars, but they are very efficient in maintaining the rooms below at an equable temperature. The clauses relating to such a roof are given in "Specification" as follows :

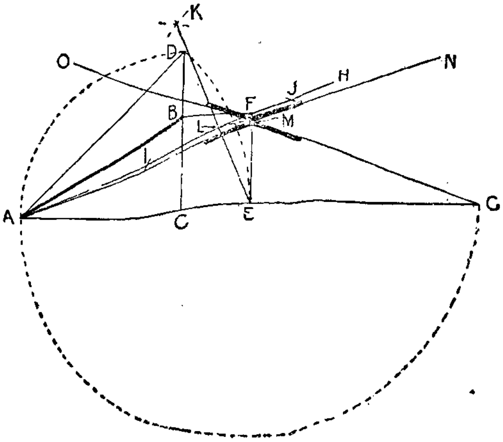


Fig. 1028.—Obtaining Bevels of Hip and Valley Tiles.

"Cover the roofs coloured . . . . on plan with . . . stone tiles, best quality, laid to a 3-in. lap at 45° pitch, with diminished courses from eaves to ridge, a double course at eaves, and dressed to hips, valleys, and verges, properly bonded in every part, and shouldered up in lime and hair mortar. The tiles to be fixed

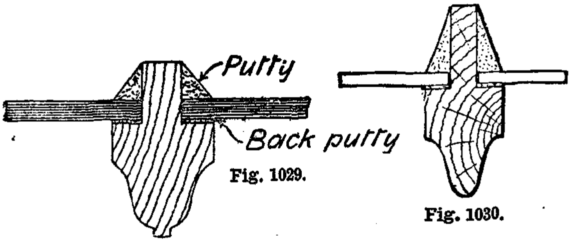

Putty
Back putty

Fig. 1029.

Fig. 1030.

Figs. 1029 and 1030.—Common Putty Glazing.

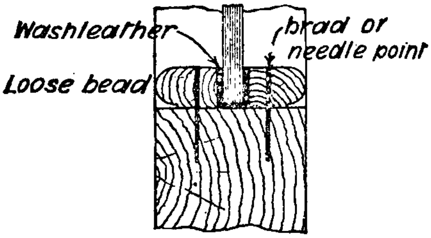
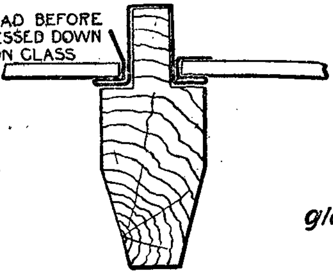
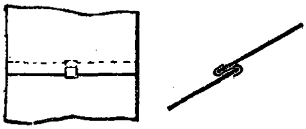

Washleather
brad or needle point
Loose bead

Fig. 1031.—Dry Glazing for Partitions and Internal Doors.


LEAD BEFORE DRESSED DOWN ON GLASS



Figs. 1032 and 1033.—Roof Glazing, held by Zinc Clips, Sides Puttied.

Fig. 1034.—Simplex Lead Glazing.

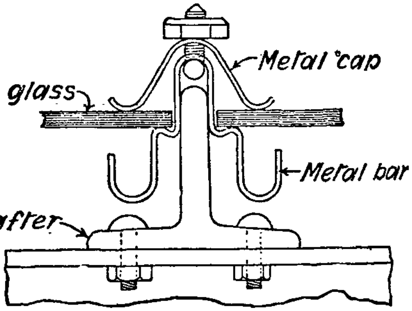

Metal cap
glass
Metal bar
Tee rafter

Fig. 1036.—Helliwell's "Perfection" Glazing on Metal Bar.

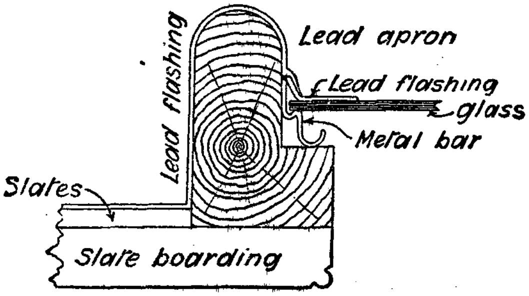

Lead flashing
Lead apron
Lead flashing
glass
Metal bar
Slates
Slate boarding

Fig. 1035.—Helliwell's "Perfection" Glazing in Slated Work.

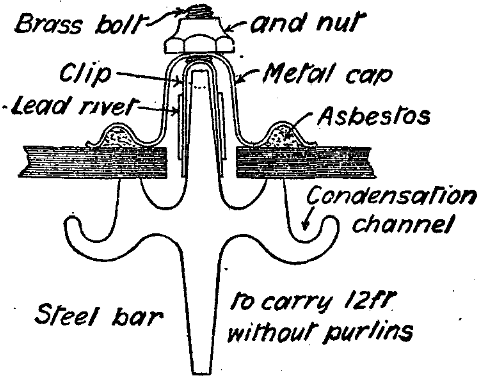

Brass bolt
and nut
Clip
Metal cap
Lead rivet
Asbestos
Condensation channel
Steel bar
to carry 12ft without purlins

Fig. 1037.—Helliwell's "Perfection" Glazing on Steel Bar.

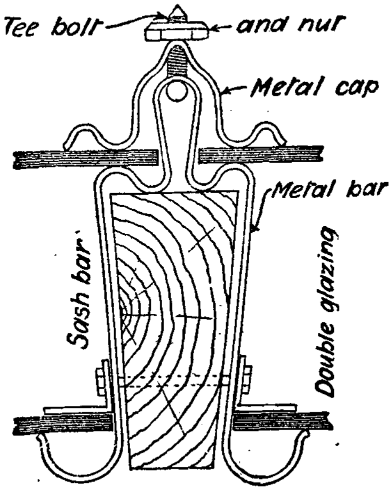

Tee bolt
and nut
Metal cap
Metal bar
Sash bar
Double glazing

Fig. 1038.—Helliwell's "Perfection" Glazing, Double.

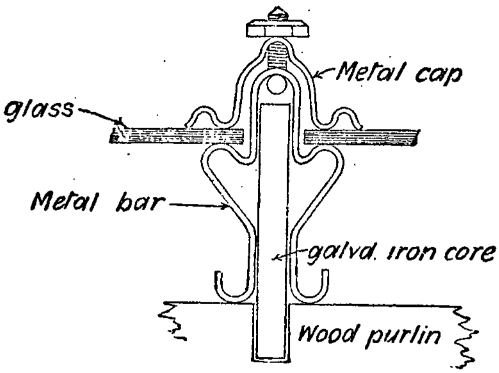

Metal cap
glass
Metal bar
galvd. iron core
Wood purlin

Fig. 1039.—Helliwell's "Perfection" Glazing on Metal Bar with Galvanised Iron Core.

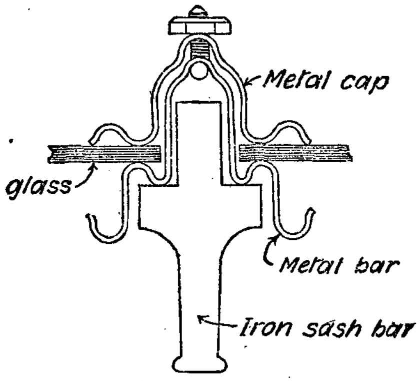

Metal cap
glass
Metal bar
Iron sash bar

Fig. 1040.—Helliwell's "Perfection" Glazing on Iron Sash Bar.

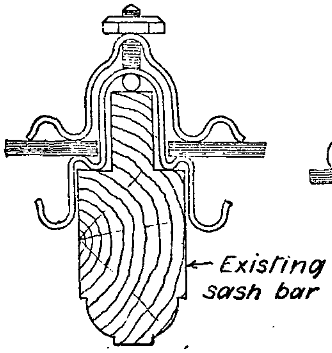

Existing sash bar

Fig. 1041.—Helliwell's "Perfection" Reglazing for Old Roof.

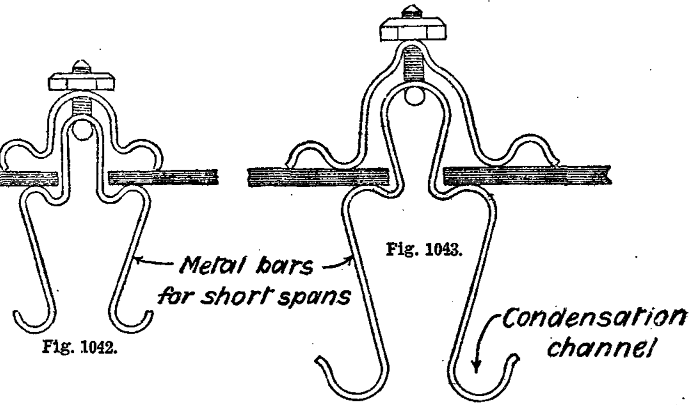

Metal bars for short spans
Condensation channel

Fig. 1042.

Fig. 1043.

Figs. 1042 and 1043. — Helliwell's "Perfection" Glazing on Metal Bars for Short Spans.

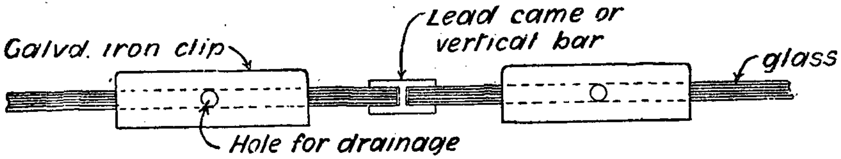

Galvd. iron clip
Lead came or vertical bar
glass
Hole for drainage

Fig. 1044.—Rendle's "Facile" Glazing.

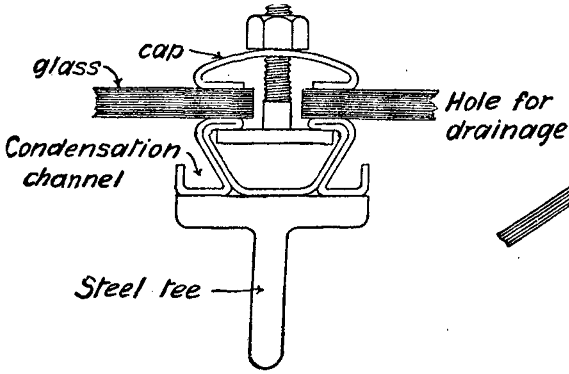

cap
glass
Hole for drainage
Condensation channel
Steel tee

Fig. 1046.—Rendle's "Invincible" Glazing on Steel Tee.

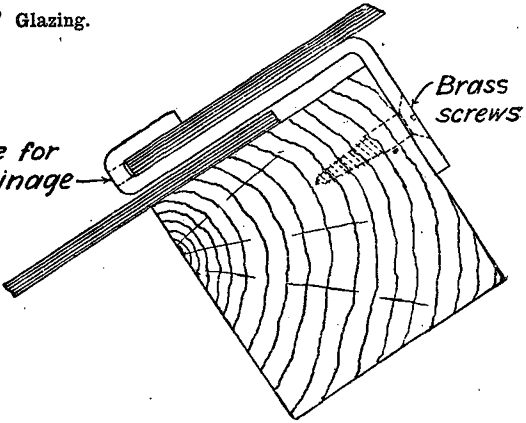

Brass screws

Fig. 1045.—Rendle's "Facile" Glazing.

with oak pegs on to 1½-in. by ½-in. sawn oak laths spiked to rafters. Perform all cutting and labour. The verges to be bedded and pointed in cement. The valleys to be formed with stone tiles worked to an angle. The hips and ridges to be formed out of solid sawn stone 9 in. wide on each splay, in lengths of not less than 3 ft. bedded and jointed in cement, with solid-cut junction pieces. The lowest hip stones to have strong wrought-iron stays." The following paragraph from "Notes on Building Construction," vol. iii., will show that such roof coverings may be called either slates or tiles, although tile stones would seem to be the best term. "Stone slates, the so-called slates being merely thin slabs of a stone that splits into thin layers along the planes of bedding, are found in various parts of the country, and are used for roofing purposes. These slates are tile stones rather than true slates. Among others may be mentioned the Collyweston and Stonesfield slates, found in several quarries of the Oolitic limestone formation near Stamford, in Lincolnshire, and Stow-on-the-Wold, in Gloucestershire. These tile stones are good non-conductors of heat, so that they keep a house cool in summer and warm in winter; but they are very heavy, especially when soaked with wet, and therefore require roofs of heavy scantlings."

Obtaining Pitch of Hip Tiles.

The pitch of hip tiles is most correctly obtained by the following geometrical construction, assuming the roof to cover the rectangular space and to be of equal pitch on all four sides. Upon the section of roof, mark any length A B (Fig. 1027) along the surface of the tiles, and draw A C E horizontal and B C D vertical, making C D = C A, and A E = A D; then draw E F vertical and B F horizontal, and join F A. Then F A C will be the pitch of hip tiles for a roof of the given pitch B A C.

Obtaining Bevels of Hip and Valley Tiles.

For roofs covering rectangular areas, and cross roofs meeting main roof at right angles, the bevels will be found as follows:—On the drawing showing section of roof mark any length A B (Fig. 1028); draw A C horizontal, and B C vertical. Produce C B to D, making C D = A C. Produce A C to E, making A E = A D. Through B draw B F horizontal, and through E draw E F vertical. Produce A E to G, making E G = A E. Join A F, producing it to any point H. From E, with any radius, cut A H in points I J, and with the same radius from centres I and J draw arcs intersecting in K. Join K E, cutting A H in point L. From E, with radius E L, draw arc L M, cutting E F in point M. Join A M and M G. Produce A M to N and G M to O. Then A M G will be the bevel for hip tiles and O M N the bevel for valley tiles, for any pitch of roof.

When the tiles are not true to required bevel, the difference is made up by mortar, which gives an unsightly appearance, and the mortar is very liable to disintegration by frost.

Manufacture and Varieties of Glass.

Glass for building uses may be divided into three chief varieties, namely, sheet glass, plate glass, and crown glass, of which the first two are in common use, while the last named has now ceased to be generally manufactured.

Sheet Glass.

Sheet glass consists of a mixture of about 60 per cent. white sand, 20 per cent. soda, and 20 per cent. chalk, mixed together and melted at a high temperature. The process of manufacture is as follows:—The man known as the "gatherer" dips the end of a long iron blowpipe repeatedly into the crucible of molten glass, until he has collected a bulb sufficiently large for his purpose. He then passes the tube to the "blower," who blows the molten glass out into a large hollow cylinder, the ends of which are cut off, and the cylinder (which is now cool) is split down one side with a diamond. The cut cylinder is next taken to the flattening kiln, which is composed of two chambers built together; the first, which is at a higher temperature than the second, being used for flattening the cut cylinders, which become heated and fall out flat by their own weight. The sheets are then passed to the second chamber, where they are placed on edge and allowed to remain for a few days to be annealed, after which they are withdrawn. Seen edgeways, this glass has a bright green colour. Sheet glass is made in many qualities, the better of which are used for pictures, and the ordinary good kinds for glazing windows, etc., while the coarsest of all is unfit for most purposes. The weight, thickness, and maximum area in feet super. in which sheet glass is made are shown in the

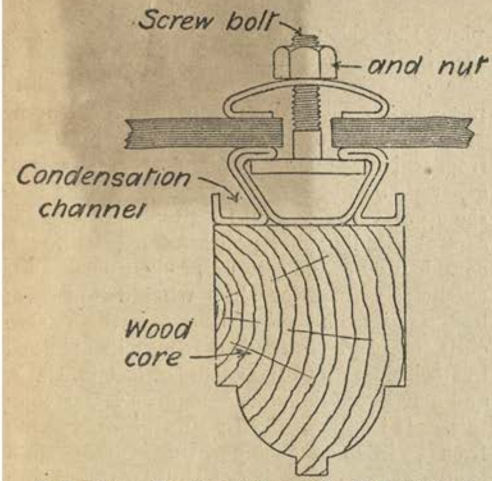

Screw bolt
and nut
Condensation channel
Wood core

Fig. 1047.—Rendle's "Invincible" Glazing on Wood Core.

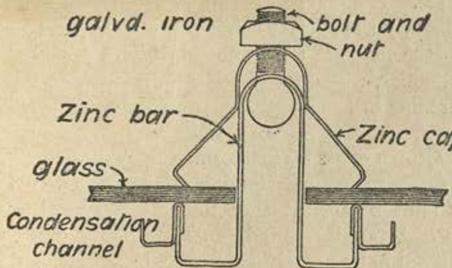

galvd. iron
bolt and nut
Zinc bar
Zinc cap
glass
Condensation channel

Fig. 1049.—Rendle's "Defiance" Glazing.

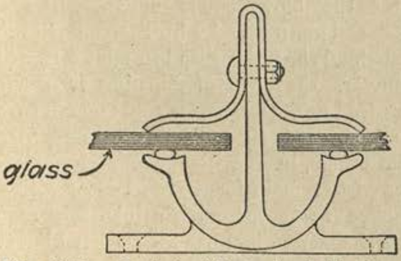


Fig. 1051.—Braby's "Drop-dry" Glazing.

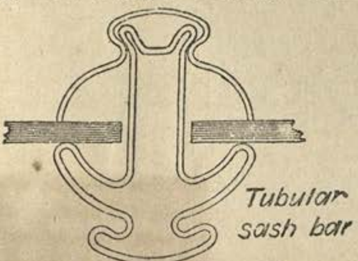

Tubular sash bar

Fig. 1053.—Braby's "Drop-dry" Glazing on Tubular Sash Bar.

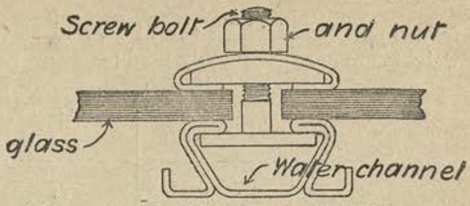

Screw bolt
and nut
glass
Water channel

Fig. 1048.—Rendle's "Invincible" Glazing, showing Water Channel.

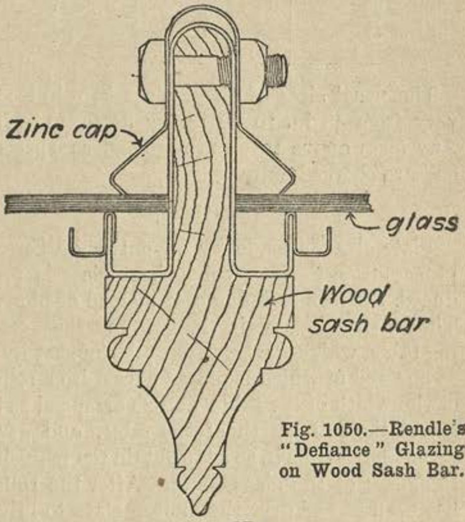

Zinc cap
glass
Wood sash bar

Fig. 1050.—Rendle's "Defiance" Glazing on Wood Sash Bar.

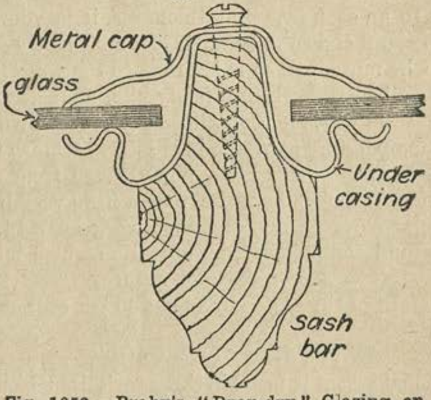

Metal cap
glass
Under casing
sash bar

Fig. 1052.—Braby's "Drop-dry" Glazing on Sash Bar.

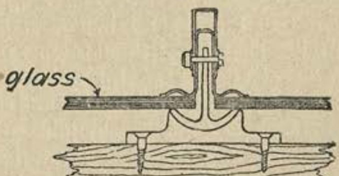


Fig. 1054.—Ellis's "Compensating" Glazing.

following table, of which the 21-oz. and 26-oz. weights are most commonly used :—

Weight in ounces per foot super.	Maximum area in feet super.	Thickness in inches.
15	13	$\frac{1}{13}$
21	22	$\frac{1}{10}$
26	22	$\frac{1}{9}$
32	22	$\frac{1}{7}$
36	19	$\frac{1}{6}$
42	19	$\frac{1}{5}$

The usual sizes of sheet glass kept in stock vary from 48 in. to 50 in. by 34 in. to 36 in. Any sizes above these are treated as special, and are charged extra.

Plate Glass.

Plate glass is made by an entirely different process from that by which sheet glass is produced, the former being cast instead of blown. The molten glass is ladled to the casting table, which is a thick flat table of cast iron, at one end of which is placed a cast-iron roller of the width of the table, and so arranged as to travel the full length of the table, and at a given height from it, so as to obtain the requisite thickness of the glass plate. After the roller has been passed over the molten glass to bring it to an even face and thickness, it is allowed to stand and solidify until just sufficiently cool to be moved, when it is taken to the annealing chamber, which is at a fairly high temperature, and the whole is then allowed to cool down to a temperature at which it is safe to remove the glass to the outer air. When taken from the annealing chamber, the plates have very irregular surfaces, and for best work have to be ground and polished, an operation which takes a good deal of time, and reduces the actual weight of the plates by about 40 per cent.

Varieties of Plate Glass.

Plate glass is made in three varieties : Rough cast plate, rolled plate, and polished plate, according to the surface. The first is the name given to the sheet taken straight from the annealing chamber without further preparation. This glass is used for many purposes where transparency is not required and strength is the main object—such as skylights, windows of railway stations, factories, lavatories, stair risers etc. In the manufacture of the rolled plate the table used, instead of being smooth, may have an indented pattern on the surface. The glass is also known as Hartley's rolled plate, being originally made under patent by Hartley & Co., of Sunderland. The material from which the polished plate glass is made is generally of a better quality than that used for the other varieties, and this, together with the careful grinding and polishing required, adds considerably to the cost of production. It is extensively used for shop windows, doors of best quality, and other uses for which large clear sheets are required. It may be obtained in stock sizes up to 100 ft. super. of $\frac{1}{4}$ in. thick, and may be specially ordered even larger than this. It is also made in thicknesses varying from $\frac{3}{16}$ in. to as much as 1 in. Patent plate glass, or blown glass, is not really plate glass at all, but is made by polishing both sides of sheet glass. It may be distinguished from British plate by the surface being more wavy, and also by the shape of the bubbles, which are oval and irregular, while those in plate glass are quite circular in form.

Crown Glass.

Crown glass is made on a blowpipe, but in a different manner from sheet glass. The mass of molten glass at the end of the blowpipe is turned rapidly round, and gradually extends outwards by centrifugal force into a large flat disc of uniform thickness, except in the centre, where the blowpipe is embedded, where a thickened lump called a bull's-eye or bullion is formed. When the disc is about 4 ft. or so in diameter, the rate of turning is gradually decreased until the glass is cool enough to retain its shape. It is then placed on edge, together with others, in the tempering chamber, and allowed to remain for a time ; after which it is removed ready for cutting into any required shape. Crown glass has now been almost entirely superseded by the better qualities of sheet glass, owing to the demand for larger sizes, and the slight increase in the demand of late years has been due to the revival of the use of small leaded light windows for middle-class dwelling-houses. This glass varies in thickness from $\frac{1}{10}$ in. to $\frac{1}{4}$ in., with a maximum stock area of about 5 ft. super., and is sold in the form of semi-discs, called tables, and also in squares or slabs. The bull's-eye transmits light and disperses it irregularly, but it does not permit of clear vision.

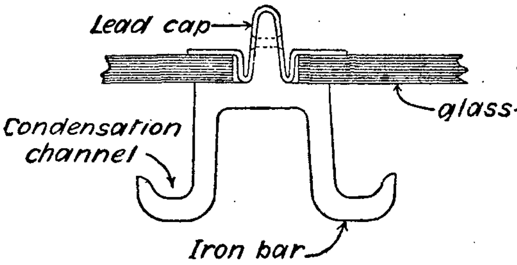

Lead cap
Condensation channel
glass
Iron bar

Fig. 1055.—Pennycook Glazing on Iron Bar.

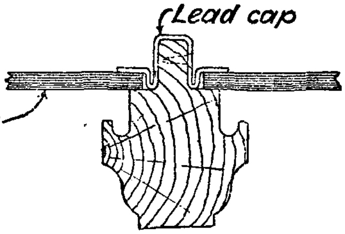


Fig. 1056.—Pennycook Glazing on Wooden Bar.

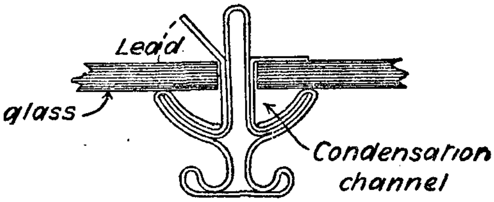

Lead
glass
Condensation channel

Fig. 1057.—Pennycook Glazing with Condensation Channel.

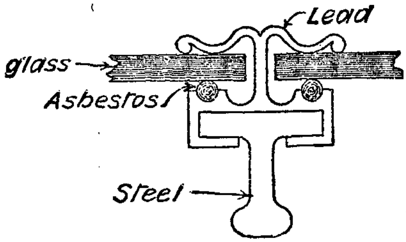

Lead
glass
Asbestos
Steel

Fig. 1058.—Heywood's Glazing.

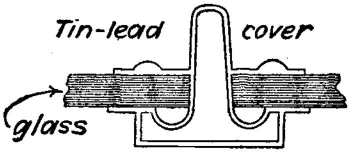

Tin-lead cover
glass

Fig. 1059.

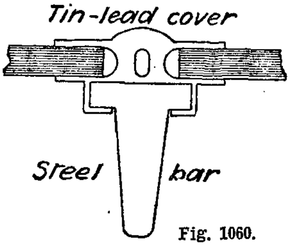

Tin-lead cover
Steel bar

Fig. 1060.

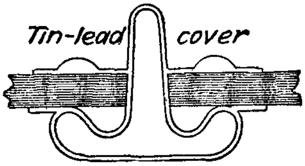

Tin-lead cover

Fig. 1061.

Figs. 1059 to 1061.—Mellowes' "Eclipse" Glazing.

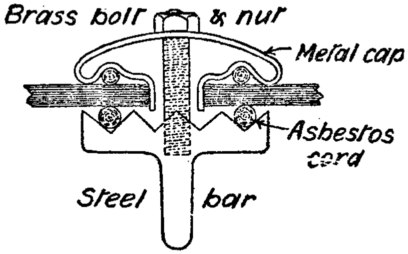

Brass bolt & nut
Metal cap
Asbestos cord
Steel bar

Fig. 1062.—Yorkshire Patent Glazing Company's Glazing.

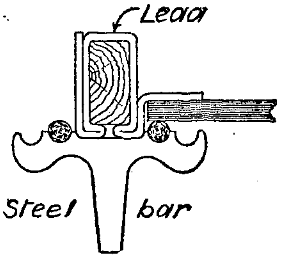

Lead
Steel bar

Fig. 1063.—Deacon's Glazing on Steel Bar.

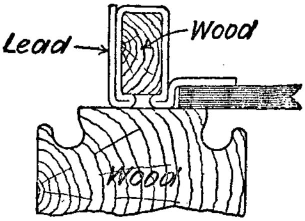

Lead
Wood
Wood

Fig. 1064.—Deacon's Glazing on Wooden Bar.

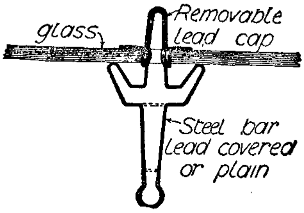

glass
Removable lead cap
Steel bar Lead covered or plain

Fig. 1065.—Deard's Glazing.

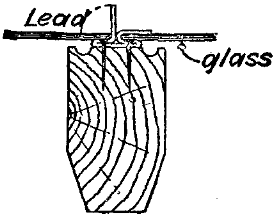


Fig. 1066.—Grover's "Simplex" Glazing.

Fancy Glass.

Ground or obscured glass is made either by placing powdered glass on one side and fluxing it in by heat, or by grinding the surface of the sheet. This glass is used for all purposes where light is required without transparency. Enamelled glass consists of powdered glass, or enamel, placed to a pattern previously stencilled on the glass, and afterwards melted in, similar to ground glass. Embossed glass is the exact opposite of enamelled glass, the surface of the glass being eaten away by hydrofluoric acid, leaving the pattern slightly raised. Coloured glass is made by adding metallic oxides, etc., to the other materials before melting, and may be divided into two varieties: (1) Flashed colours in which ordinary sheet glass is covered with a thin film of coloured glass; and (2) those in which the colour is constant through the entire thickness. In the former, designs may be obtained by eating off the surface colouring with hydrofluoric acid, leaving the transparent portion underneath fully exposed.

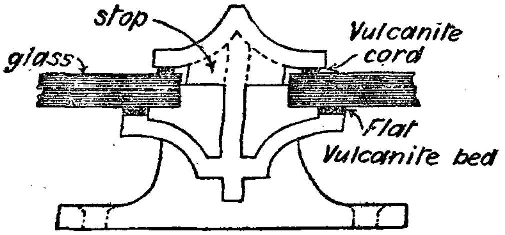
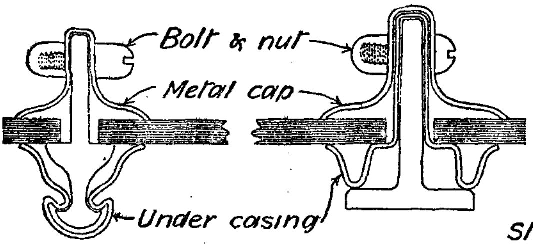


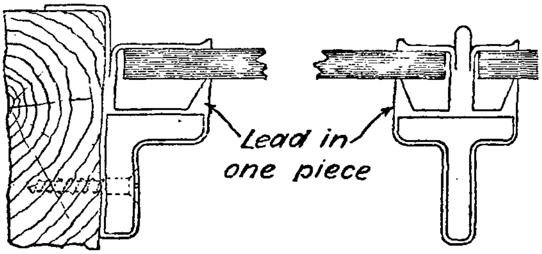
Fig. 1067.—T. R. Shelley's Glazing.

Glass slates and tiles are made of either sheet or rolled glass, and may be obtained in any tile and slate size and shape. They are worked in the roofs of buildings where light is required without going to the expense of forming skylights.



Figs. 1068 and 1069.—Shelley & Co.'s "Unique" Glazing.

There are also many other varieties of glass, such as flint or crystal glass, bottle glass, optical glass, spun glass, hardened glass, etc.; but these are so rarely connected with the building trades that any account of their composition and manufacture would be out of place here.

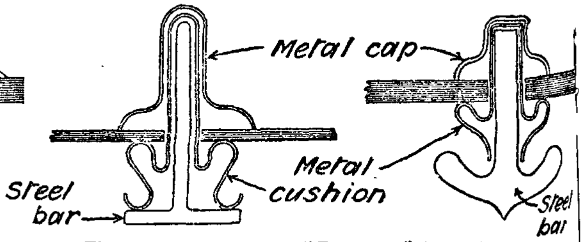


Figs. 1070 and 1071.—British Challenge Glazing.

Fire-Resisting Glass.

The disastrous Cripplegate fire in the City of London in 1897 called attention to the danger arising from unprotected glass in windows falling out at the first breath of flame, and leaving a large area exposed for the passage of fire. To obviate this danger several forms of protected glass have been put on the market.

The "Electro-Glass," or wired glass, made by The British Luxfer Prism Syndicate (16, Hill Street, London, E.C.), remains in place until the sash is burnt away, or the glass itself melted, and is therefore a most valuable screen for preventing the spread of fire through window and door openings. It is suitable for enclosing liftways, stairways, etc., as well as for ordinary glazing. The retail price is about 3s. per ft. super.



Figs. 1072 and 1073.—"Paragon" Patent.

Patent Wired Glass made by Pilkington Bros., Limited (St. Helen's, Lancashire), is very largely used. It has wire netting enclosed in sheets up to 110 in. long by 36 in. wide, at about 1s. per ft. super., and at slightly increased prices for larger dimensions. The standard thickness is ¼ in. It is claimed to be fire-proof, burglar-proof, and stone-proof, and is particularly useful for roof work.

"Besto" Glass, made by the Besto Glass Company (41, Eastcheap, London, E.C.), consists of ordinary wire netting covered with asbestos embedded and wholly enclosed in the body of the glass, which is either white, coloured, transparent, translucent, or opaque, and from ¼ to 1 in. thick, according to the purpose for which it is required. It is primarily intended for use in skylights, roofs, partitions, floors, fanlights, doors, windows, awnings, etc.; in fact, in any position where the heavy glass known as "rough rolled plate," or other glass in plates, sheets, blocks, or prisms, is commonly used. The sheets are manufactured up to 3 ft. × 10 ft. The Besto Prism Plate is one of the patterns most in favour in the United States; it is smooth on one side, and on the other has longitudinal prismatic ribs each about ½ in. in width.

"Wired Refrax Glass" is made by the Union Plate Glass Company, Limited (Pocket Nook, St. Helen's), in all sizes, about ¼ in. thick, is said to have all the advantages of the others, and to transmit 100 per cent. more daylight.

Siemens' Wired Glass is good, but not obtainable in England.

The "Williams" Fire Blinds, made of asbestos cloth and hung on rollers, afford good protection against the passage of flame to ordinary doors and windows.

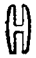


Fig. 1074.—Section of Lead Came.

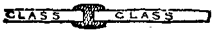


Fig. 1075.—Section of Glass in Lead Came.

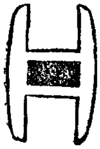


Fig. 1078.—Section of Lead Came with Steel Centre.

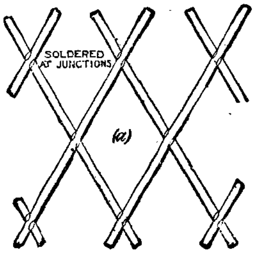


Fig. 1076.—Lead Cames soldered at Junctions.

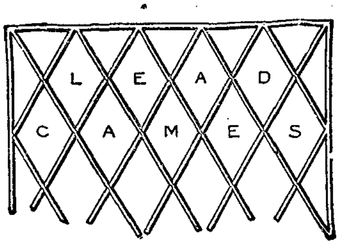


Fig. 1077.—Finished Lead Cames.

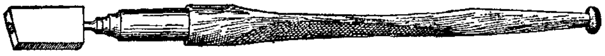


Fig. 1079.—Glazier's Diamond.

Modes of Glazing.

The varieties of glazing are more easily described in illustrations than in words, and the sectional views about to be referred to will be found to give all necessary particulars. Common glazing with putty is shown by Figs. 1029 and 1030; dry glazing for partitions and internal doors by Fig. 1031; and roof glazing held by zinc clips and having the sides puttied by Figs. 1032 and 1033. There is a great number of special and patent systems, these including "Simplex" lead glazing (Fig. 1034); Helliwell's "Perfection" glazing (Figs. 1035 to 1043); Rendle's "Facile" glazing (Figs. 1044 and 1045); Rendle's "Invincible" or "Acme" glazing (Figs. 1046 to 1048); Rendle's "Defiance" glazing (Figs. 1049 and 1050); Braby's "Drop-dry" glazing (Figs. 1051 to 1053); Ellis' "Compensating" glazing (Fig. 1054); Pennycook glazing (Figs. 1055 to 1057); Heywood's glazing (Fig. 1058); Mellowes "Eclipse" glazing (Figs. 1059 to 1061); Yorkshire Patent Glazing Company's glazing (Fig. 1062); Deacon's glazing (Figs. 1063 and 1064); Deards' glazing (Fig. 1065); Grover's "Simplex" lead glazing (Fig. 1066); T. R. Shelley's glazing (Fig. 1067); Shelley and Company's "Unique" glazing (Figs. 1068 and 1069); the British Challenge Glazing Company's glazing (Figs. 1070 and 1071); and "Paragon" patent glazing (Figs. 1072 and 1073). In every case the section of bar must be of a strength suitable to the span. These patent glazings are mostly used for large conservatories, market roofs, and railway stations.

Leaded Lights.

Illustrations showing how glass is secured in lead cames are presented by Figs. 1074 to 1077. The came is shaped in section as in Fig. 1074, the glass being inserted as in Fig. 1075. The cames are soldered together at their junctions as in Fig. 1076, the appearance of the finished cames being as indicated in Fig. 1077. Lead glazing is naturally weak, and is usually supported by saddle bars, to which the cames are attached by means of small copper wire soldered to the cames and bent round the saddle bar. An important improvement has recently been introduced, consisting of a small steel bar rolled in the centre of the came as shown enlarged in Fig. 1078. The steel being edgeways to the pressure renders the glazing very stiff, and saddle bars may be dispensed with.

Glazier's Diamond.

The glazier's diamond (Fig. 1079) is used for cutting and breaking glass. It is held with the thin part of the handle between the first and second fingers of the right hand, and also by the thumb and tips of the fingers, the first finger being placed upon the brass mounting carrying the diamond to give the required pressure. The angle at which it is held is such that the bottom of the mounting is parallel with the glass. When properly held to suit the particular diamond, only a slight pressure is needed, and then by shifting the glass to the edge of the cutting board and tapping the overhanging portion a true edge is made.

IRON, STEEL, AND FIREPROOF CONSTRUCTION.

Ironwork.

In speaking of ironwork it should be remembered that iron is the generic term which includes cast iron, wrought iron, and steel; but when one only of these subdivisions is intended, it should be specified. Constructional ironwork is therefore a proper general term, although in modern usage it would seem to exclude steel, as many persons look upon steel as a different metal from iron, instead of its being only manufactured by a different process.

Sections of Cast and Rolled Girders.

Figs. 1080 and 1081 show the ordinary sections adopted for cast and rolled iron girders respectively. In a cast-iron girder the top flange is one-fourth the sectional area of the bottom flange. The rolled iron joist (Fig. 1081) weighs 54 lb. per foot run, and the flanges are of equal sectional area. A typical rolled joist is shown by Fig. 1082, and three kinds of compound girders by Figs. 1083 to 1085. Twin joists with cast-iron distance pieces between are illustrated by Figs. 1086 and 1087, a distance piece being shown by Fig. 1088.

Plate Girders.

A cross section of a wrought-iron built-up girder is presented by Fig. 1089, and part elevation by Fig. 1090. The rivets are at a 4-in. pitch. The section and elevation of a plate iron box girder, showing the rivets, are given by Figs. 1091 and 1092. The section is at a point where a joint occurs in the upper flange, but the elevation is of a part where there is no joint in the flanges, and shows the rivets at a 4-in. pitch. The flange plates are increased in number towards the centre of a plate girder because the bending moment is greatest there, and the bending moment is the measure of the longitudinal stress in the flanges. In a girder carrying a uniformly distributed load and supported at the ends the bending moment varies as ordinates to a parabola, and the plates are cut off just beyond the outline of this curve, as shown in Fig. 1094. In designing girders the parabola is drawn from centre to centre of bearing surfaces to any scale, with a height equal to the calculated thickness of the flange in the centre, and generally drawn full size in this direction. The web is reduced in thickness towards the centre, because the shearing stresses are least there and greatest at the abutments. The plates are regulated in thickness by drawing a sectional plan as shown in Fig. 1093 to similar scale as Fig. 1094, calculating the thickness required at the abutments and drawing triangles as shown. The plates are then reduced so as to include everywhere the triangles, and no plates are less than ¼ in. thick, or ⅜ in. where the girder exceeds, say, 3 ft. in depth.

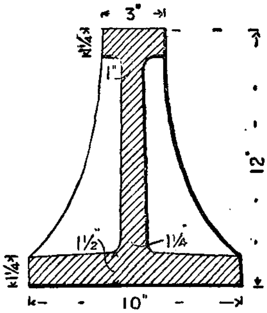


Fig. 1080.—Section of Cast-iron Girder.

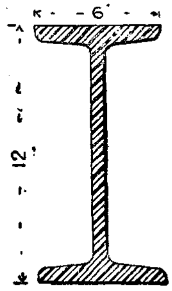


Fig. 1081.—Section of Rolled Iron Joist.

Testing Rolled Joists and Bars.

An inspector usually tests about 5 per cent. of the material for weight, and a variation not exceeding 2½ per cent. is allowed. About 5 per

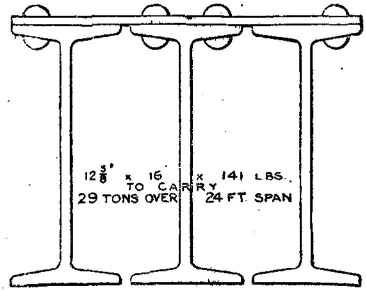

12⅜" × 16" × 141 LBS.
TO CARRY
29 TONS OVER 24 FT. SPAN

Fig. 1083.—Compound Girder.

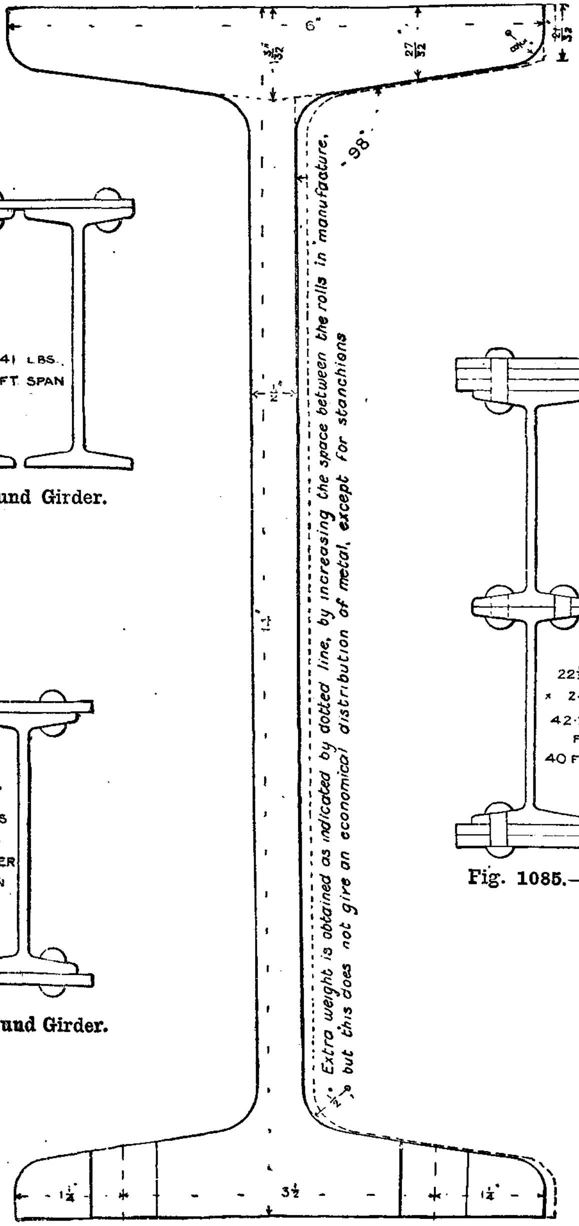

6"
1 3/32"
27/32
21/32
·98"
½"
Extra weight is obtained as indicated by dotted line, by increasing the space between the rolls in manufacture,
but this does not give an economical distribution of metal, except for stanchions
½"
1¼"
3½
1¼"

Fig. 1082.—Typical Section of Rolled Joist.

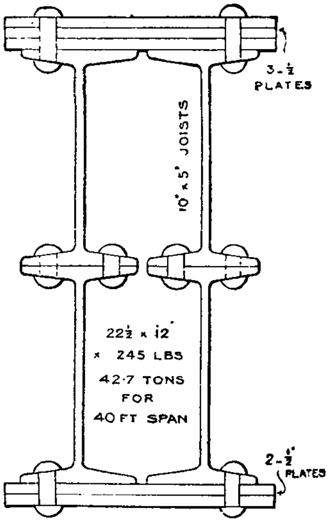

3-½ PLATES
10" × 5" JOISTS
22½ × 12"
× 245 LBS
42·7 TONS
FOR
40 FT SPAN
2-½" PLATES

Fig. 1085.—Compound Girder.

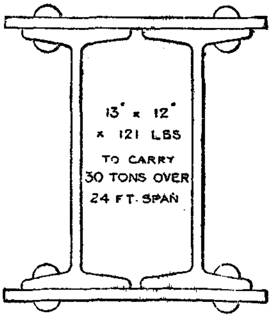
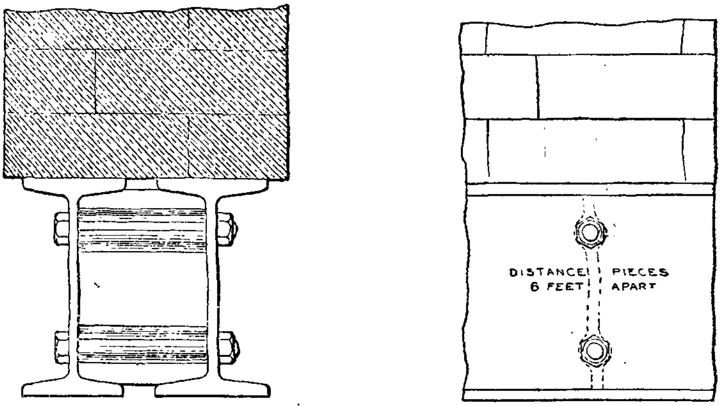

13" × 12"
× 121 LBS
TO CARRY
30 TONS OVER
24 FT. SPAN

Fig. 1084.—Compound Girder.

cent., or not less than one bar of each size, is tested for tensile strength, the pieces being taken from the ends. The elastic limit, ultimate strength, elongation, etc., will depend on the specification. With regard to the detection of foreign rolled joists it may be said that foreign joists are frequently in odd sizes, the dimensions being in millimetres instead of inches (or quarter-inches in the width of some of the smaller joists); but the foreign dimensions may in some cases be so near to English sizes that the two cannot be distinguished from each other. The joists are also in some cases made purposely to English sizes in order to undersell the market. In some cases the web may be thickened abnormally in order to make up the weight; but abnormal thickness is practically useless in this position, and the joist, therefore, will be considerably weaker than an English joist of the same weight and tensile strength. The larger sizes of foreign joists are more often than in the case of English joists made up of portions welded together along the web, and this may be seen on examining the ends. Rolled joists usually carry the maker's name and trade mark in raised characters rolled on the web; besides which, joists made abroad must, when imported to this country, be marked with the country of origin, as, for example, "made in Belgium." If a short piece of joist only is available for examination, the trade mark may be absent, but if the joist is of Belgian make the rounded edges will probably be squarer than is usual in English sections. Foreign joists are rather apt to be twisted slightly, to have the flanges buckled and the depth not uniform when with welded webs. Rolled joists cannot be tested on the site as regards strength, except by loading to destruction. If the maker be well and honourably known the quality, etc., of the material may be relied upon, without testing. An inspection of the punchings or drill shavings and of the behaviour of the material under the chipping chisel, will tell a good deal to a practised eye.



Figs. 1086 and 1087.—Twin Joists with Cast-iron Distance Pieces between.

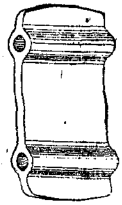
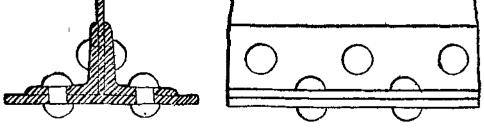


Fig. 1088.—Distance Piece.

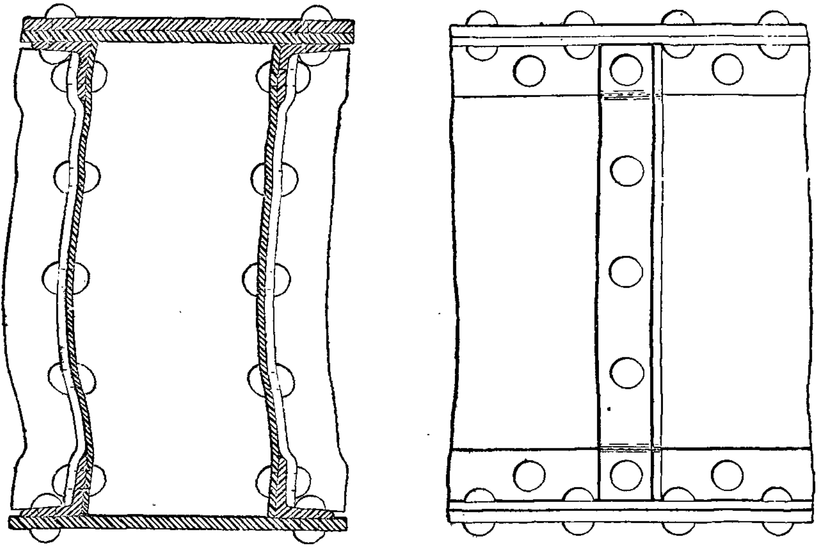
Weight of Joists.

The weight in lb. per foot run of a rolled steel joist is approximately 3⅓ times the sectional area in square inches. Assuming the flanges to average ⅜ in. thick, and the web to be ⅜ in., the sectional area of the joist shown in Fig. 1095 will be $2 \times 3 \times \frac{3}{8} + 2\frac{1}{4} \times \frac{3}{8} = 8\frac{1}{4} \times \frac{3}{8} = 3{\cdot}09375$ sq. in. This multiplied by $3\frac{1}{3} = 10{\cdot}3125$ lb. per foot run for the required weight.



Figs. 1089 and 1090.—Built-up Plate Girder.

The standard section for 3-in. by 3-in. joist is 10 lb. per foot run, the web being only $\frac{19}{64}$ in. thick instead of $\frac{3}{8}$ in. The approximate weight of a girder of any kind should be taken into account as part of the load the girder has to support. Let W = safe distributed load in tons, L = clear span in feet, d = total depth (more correctly mean depth) in inches, w = weight of girder. Then for rolled steel joists, w lb. per foot run $= 2\frac{\text{W L}}{d}$; wrought-iron plate girder or steel box girder, w in tons $= \frac{\text{W L}}{375} \times \sqrt{\frac{\text{L}}{d}}$; steel plate girder, w in tons $= \frac{\text{W L}}{400} \times \sqrt{\frac{\text{L}}{d}}$. Steel, although stronger than wrought iron, is slightly heavier.



Figs. 1091 and 1092.—Plate Iron Box Girder.

Concentrated Loads and Distributed Loads.

Comparisons of strength according to the distribution of the load and the mode of supporting the joists are afforded by Figs. 1096 to 1103, the first four figures indicating concentrated loads, and the second four figures distributed loads.

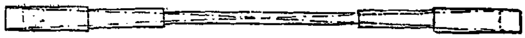


Fig. 1093.—Sectional Plan of Web Plates.

Distributed Load on Joist.

Assume a rolled steel joist of the section shown in Fig. 1104 placed on bearings 12 ft. apart; it is required to know what distributed load it will carry approximately, the stress in it being limited to 4 tons per square inch. Area $= 2 \times 6 \times \frac{3}{4} + 8\frac{1}{2} \times \frac{1}{2} = 13\frac{1}{4}$ sq. in. Weight per foot run $= 13\frac{1}{4} \times 3\frac{1}{3} = 44{\cdot}16$, say 45 lb. Maximum bending moment under uniformly distributed load

$$\bar{\text{M}} = \frac{\text{W L}}{8}$$

and maximum stress in each flange $= \frac{\text{W L}}{8\,d}$, but flange $= 4\frac{1}{2}$ sq. in., and safe stress = 4 tons per sq. in. $\therefore \frac{\text{W L}}{8\,d} = 4\frac{1}{2} \times 4$, whence W =

$$\frac{8\,d \times 4\frac{1}{2} \times 4}{\text{L}} = \frac{8 \times 10 \times 4\frac{1}{2} \times 4}{12 \times 12} = 10 \text{ tons.}$$

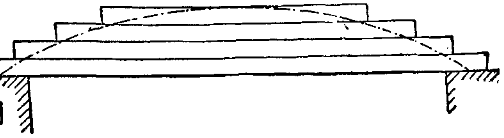


Fig. 1094.—Flange Plates cut off to Parabola Outline.

In the above formula, L being in feet and d in inches, L is multiplied by 12 to convert it to inches.

Another method:

f = stress allowed, tons per square inch
L = span in feet
a = area one flange in square inches
d = depth of joist in inches

$$\text{W} = \frac{f\,a\,d}{1\frac{1}{2}\,\text{L}} = \frac{4 \times 4\frac{1}{2} \times 10}{1\frac{1}{2} \times 12} = 10 \text{ tons}$$

To solve this problem accurately the moment of inertia or modulus of section should be taken, and the mean span centre to centre of bearing surfaces, allowance being made for the weight of the rolled joist itself. The section referred to above is Dorman, Long & Co.'s No. *G 9 Section, 45 lb. per foot run and 13·23 sq. in. total area. Modulus of section 43·19. Safe load at 8 tons per square inch maximum stress, or $\frac{1}{4}$ breaking load = 230·36 tons on a span of 1 ft., ∴ for 12-ft. span $\frac{230{\cdot}36}{12}$ = 19·19, say 20 tons, or half this = 10 tons, with a maximum stress of 4 tons per square inch. The exact agreement of the three methods is accidental; if another size of joist had been selected the results would probably have differed more or less.

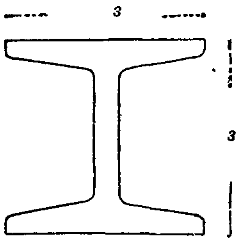


Fig. 1095.—Section of Rolled Steel Joist.

Proportions of Rolled Joists.

Rolled joists are now practically all mild steel, as there is no object in continuing the manufacture in wrought iron. Certain sections have been approved by the Engineering Standards Committee, but the list is too long for quotation here. A copy of the list, price 1s., may be obtained from 28, Victoria Street, S.W., or Crosby Lockwood and Son, Stationers' Hall Court, London, E.C. The proportions of an average section may be taken as—width of flange = half depth of joist, thickness of flange half-way between edge and web = $\frac{1}{8}$ of width, thickness of web = $\frac{2}{3}$ thickness of flange, angle between flange and web = 98°. The formula for calculating an iron girder is as follows:

Let W = breaking weight centre tons.
a = area bottom flange square inches.
d = depth of girder in inches.
c = constant.
L = span in feet.

Formula: $\text{W} = \frac{adc}{\text{L}}$. The following constants should be noted: $c = 2$ cast iron, 6 wrought iron riveted, 7 wrought iron rolled solid, 6 compound ditto, 10 rolled steel joists, 9 compound girders of rolled steel with plates.

Girder Fixed at Both Ends.

When a girder with a concentrated load is merely supported at the ends, it bends downwards in one curve over the whole span, and there is only one section where it would break if loaded to the utmost. When a girder is fixed at both ends there is a similar bend in the centre, but, the ends not being free to rise, a reverse curve is caused there, making three places where fracture could take place at the same time if loaded sufficiently. So that the girder would seem to be three times as strong when fixed at the ends; but, as a matter of fact, it will only bear twice as much if the section is a solid rectangle, while if it were cast iron of the usual section it would be weaker when fixed at the ends. A beam built in at the ends is under the same conditions as the Forth Bridge, and is similar to a beam of half the span resting upon two cantilevers, each of one-fourth the span, built in the walls. A cast-iron girder should not be so fixed at the ends as to become equivalent to one bay of a continuous girder; no saving in weight is obtained, and so many precautions have to be taken that the advantages are problematical. The span of a fixed beam is from centre to centre of the lower bearing surfaces, but the beam is not fixed unless proper top-bearing surfaces are provided farther back.

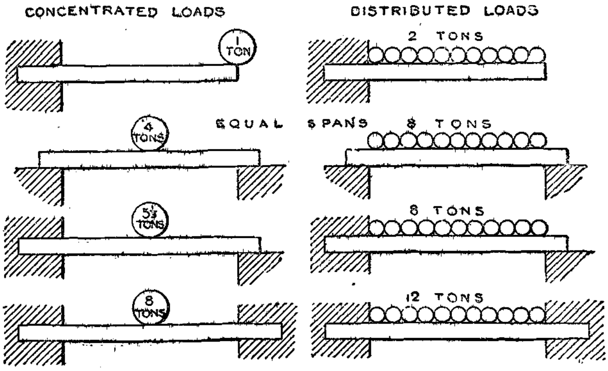
Stresses in Rolled Joist.

In a rolled iron beam supported at both ends and carrying a uniformly distributed load the stresses set up are:—Top flange subject to compression, varying from a maximum in the middle to nil at the ends. Bottom flange subject to tension, varying from a maximum in the middle to nil at the ends. Web subject to a vertical shearing stress, varying from a maximum at the ends to a minimum at the middle.

Also an equal horizontal shearing stress varying from the neutral axis to the flange.

Determining Size of Rolled Steel Joist.

It is rather a difficult matter to arrive at the proper size of a rolled steel joist by calculation when the load and span are given. It is much simpler to refer to a reliable catalogue, where the loads are all tabulated for the different sizes of joists ; but there is, of course, a risk in taking the figures given unless they are known to be correct. Assuming a load of 30 tons and a span of 12 ft., the ordinary formula for strength of rolled steel joist is $\text{W} = \frac{10\,a\,d}{\text{L}}$ where W = breaking weight in tons in centre, a = net area in square inches of one flange, together with one-sixth of the web, d = depth in inches, L = span in feet. But a girder will carry double the load if distributed, and the working load should be, say, one-fourth of the breaking weight ; then making W′ = safe distributed load, the formula becomes $\text{W}' = \frac{10\,a\,d}{\text{L}} \times \frac{2}{4}$, or $\text{W}' = \frac{5\,a\,d}{\text{L}}$, whence $a = \frac{\text{W}'\text{L}}{5\,d}$. As the load in this case will prevent the depth of joist being made the usual proportion of $\frac{1}{24}$ span = 6 in., a trial may be made of 12 in. deep, then $a = \frac{30 \times 12}{5 \times 12} = 6$ sq. in. This will be given approximately by a 12-in. by 6-in. by 54-lb. rolled steel joist ; or a 10-in. by 8-in. by 70-lb. rolled steel joist will be nearer, but will be more costly on account of its greater weight.



Figs. 1096 to 1103.—Diagrams illustrating Loading of Beams.

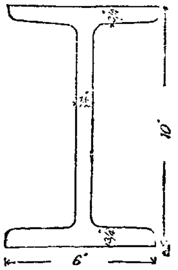


Fig. 1104.—Section of Rolled Steel Joist.

Determining Size of Built-up Steel Girder.

Assume that it is required to obtain the size of a built-up steel girder that would bear a weight of 81 tons over an 18 ft. opening; further assuming the mean depth to be one-twelfth of the clear span, a span of 18 ft. will give a depth of 18 in. The weight of the girder must be taken into account in calculating the strength. The approximate weight of a steel box girder is $\frac{\text{W}\,\text{L}}{375} \times \sqrt{\frac{\text{L}}{d}}$, where W = external distributed load in tons, L = clear span in feet, d = mean depth in inches ; then the weight of the girder $= \frac{81 \times 18}{375} \times \sqrt{\frac{18}{18}} = 3{\cdot}888$, say 4 tons, making the total load 85 tons. The maximum bending moment under a distributed load is $\frac{\text{W}\,\text{L}}{8}$; but L is now the effective span measured from centre to centre of the bearing surfaces, say 20 ft., so $\frac{\text{W}\,\text{L}}{8} = \frac{85 \times 20}{8} = 212{\cdot}5$ ton-ft. Dividing this by the mean depth in feet gives the stress in the flange $\frac{212{\cdot}5}{1{\cdot}5} = 141{\cdot}7$, say 142 tons. The working stress allowed may be 7½ tons per square inch on the gross section. Then $\frac{142}{7{\cdot}5} = 18{\cdot}93$, say 19 sq. in. If the girder is 12 in. wide the mean thickness of the flange must be $\frac{19}{12} = 1{\cdot}583$, say 1⅝ in. If the girder is made up of rolled joists, with top and bottom plates, Dorman, Long & Co.'s Compound Girder G5C5, 18 in. by 12 in. by 210 lb. per ft.

may be used. This is made up of two 15-in. by 5-in. by 42-lb. rolled steel joists, and six 12-in. by ½-in. plates.

Designing Girders and Columns.

There is no simple method of obtaining the proper sizes for girders and columns in a large building. The student will find that, although approximate rules may be laid down for certain cases, so many points have to be considered in designing that the work is usually referred to an expert if the case is important, or left to the manufacturer if simple and straightforward. Roughly, the net sectional area of the flange of a wrought-iron plate girder is three-eighths of the distributed load in tons. The area of the bottom flange of a cast-iron girder in square inches = the distributed load in tons. For rolled steel joists with depth in inches = ½ span in feet, twice the load in tons distributed × span in feet ÷ depth in inches gives the weight in pounds per foot run. Cast-iron columns and stanchions average proportions up to fifteen diameters long will carry four tons for every square inch sectional area. Built-up steel stanchions up to thirty diameters long may be loaded to 3¼ tons per square inch. These figures must only be looked on as giving a general idea of the requisite sizes. In practice every case should be taken on its own merits.

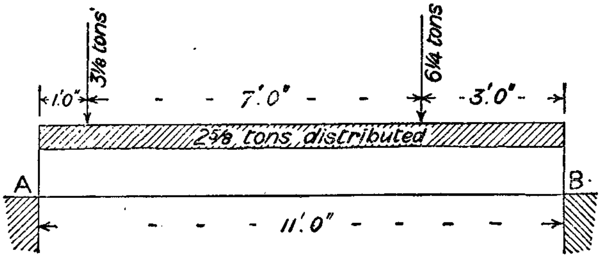


Fig. 1105.—Loaded Girder.

Determining Size of Girder to meet Special Requirements.

Fig. 1105 shows a case in which a girder is required to carry weights as indicated, and an additional uniformly distributed dead load of 2⅝ tons—total load, 12 tons. The first thing is to find the reaction at each end. At end A reaction from distributed load $= \frac{2\frac{5}{8}}{2} = 1{\cdot}3125$ tons, from 3⅛-ton load $= \frac{3\frac{1}{8} \times (7 + 3)}{11} = 2{\cdot}841$ tons, and from 6¼-ton load $\frac{6\frac{1}{4} \times 3}{11} = 1{\cdot}705$ tons. Together $= 1{\cdot}3125 + 2{\cdot}841 + 1{\cdot}705 = 5{\cdot}858$ tons. As a check upon the working, the reaction at B should be found in the same way. From distributed load $= \frac{2\frac{5}{8}}{2} = 1{\cdot}3125$ tons, from 3⅛-ton load $= \frac{3\frac{1}{8} \times 1}{11} = 0{\cdot}284$ ton, from 6¼ ton load $= \frac{6\frac{1}{4} \times (7 + 1)}{11} = 4{\cdot}5454$ tons. Together $= 1{\cdot}3125 + 0{\cdot}284 + 4{\cdot}5454 = 6{\cdot}142$ tons. The sum of the reactions $5{\cdot}858 + 6{\cdot}142 = 12$ tons, which agrees with the sum of the loads $3\frac{1}{8} + 6\frac{1}{4} + 2\frac{5}{8} = 12$ tons. The next step is to find the greatest bending moment, and this can be very easily done by a combination of graphic diagram and calculation as in Fig. 1106. The ends of the girder are merely built into the wall, and are not fixed in the scientific sense of the word; merely building in the ends of a girder does not fix it so as to alter the stresses. In Fig. 1106 the parabola above the line shows the bending moments produced by the distributed load, the maximum ordinate being $\frac{\text{W L}}{8} = \frac{2\frac{5}{8} \times 11}{8} = 3{\cdot}609$ ton-ft. The 3⅛-ton load produces a maximum bending moment of $\frac{\text{W}ab}{\text{L}} = \frac{3\frac{1}{8} \times 1 \times (7 + 3)}{11} = 2{\cdot}84$ ton-ft. directly under the load and throughout the remainder of beam as shown by triangle. The 6¼-ton load produces a maximum bending moment of $\frac{\text{W}ab}{\text{L}} = \frac{6\frac{1}{4} \times 3 \times (7 + 1)}{11} = 13{\cdot}636$ ton-ft. directly under the load, and throughout remainder of beam as shown by triangle. The two triangles are seen to overlap, and one portion must be transferred to the outside, making the polygonal outline shown. The whole shaded area now gives the bending moments from end to end of girder; the maximum being scaled off is found to be 17·45 ton-ft. under the $6\frac{1}{2}$-ton load. Then eight times the maximum bending moment in ton-ft. is the distributed load in tons that 1 ft. will carry according to the tables in Dorman, Long and Co.'s catalogue. Thus $\frac{\text{W L}}{8} = \text{M}, \therefore \text{W} = \frac{8\text{ M}}{\text{L}}$, but $\frac{\text{dist. load 1 ft.}}{\text{span}} = \text{W}, \therefore \frac{8\text{ M}}{\text{L}} = \frac{\text{dist. load 1 ft.}}{\text{span}}$ or cancelling, 8 M = dist. load 1 ft. will carry.

Deflection of Girders.

The deflection of a steel flanged girder of uniform strength, supported at both ends and carrying a uniformly distributed dead load, may be found approximately by the formula $\text{D} = \frac{\text{K}\,l}{8\,d}$, $k = \pm\frac{\text{S L}}{\text{E}}$, $\text{K} = 2\,k$, where D = central deflection in inches, d = central mean depth in inches, l = effective span in inches, K = sum of the extension of one flange or boom and the shortening of the other as the result of strains, S = stress in pounds per square inch on either boom when the load producing the deflection is on the beam, E = modulus of elasticity in pounds, L = length of boom in inches, and k = extension or compression of the boom after the strain is on.

Stiffeners to Rolled Joists.

In order to find whether a rolled joist requires stiffeners, work as follows: Take, for example, a 20-in. by $7\frac{1}{2}$-in. by 89-lb. rolled steel joist carrying a distributed load of 36 tons. ∴ Reaction at each end $= \frac{36}{2} = 18$ tons. Permissible shear stress in tons = 5 − $\frac{1}{6}$ depth $= 5 - \frac{20}{6} = 1{\cdot}67$ tons. Actual depth of web = 20 − 2 flanges $1\frac{1}{4}$ in. each = 17·5 in. Net depth of web 17·5 − $\frac{1}{3}$ of 17·5 taken as part of flanges = 11·7 in. Shear stress per inch in depth $= \frac{\text{reaction}}{\text{net depth}} = \frac{18}{11{\cdot}7} = 1{\cdot}54$ tons. Required thickness of web = $\frac{\text{shear stress per inch in depth}}{\text{permissible stress per square inch}} = \frac{1{\cdot}54}{1{\cdot}67} = {\cdot}92$ in. The actual thickness is only $\frac{13}{32} = {\cdot}4$ in., therefore stiffeners will be required at the abutments and at the ends.

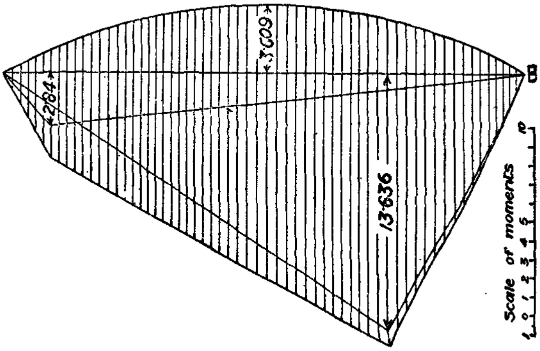


Fig. 1106.—Graphic Diagram to Determine Greatest Bending Moment.

Designing Plate Girder for Given Load and Span.

It is required to draw the central section of a plate girder to carry 2 tons per foot run over a 30-ft. span, the depth at the centre being 30 in., and the tensile stress on the metal, due to the load, being limited to 5 tons to the inch. Formula for wrought-iron plate girder:—

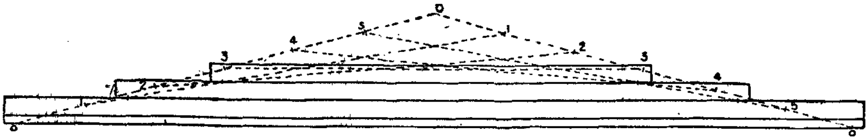
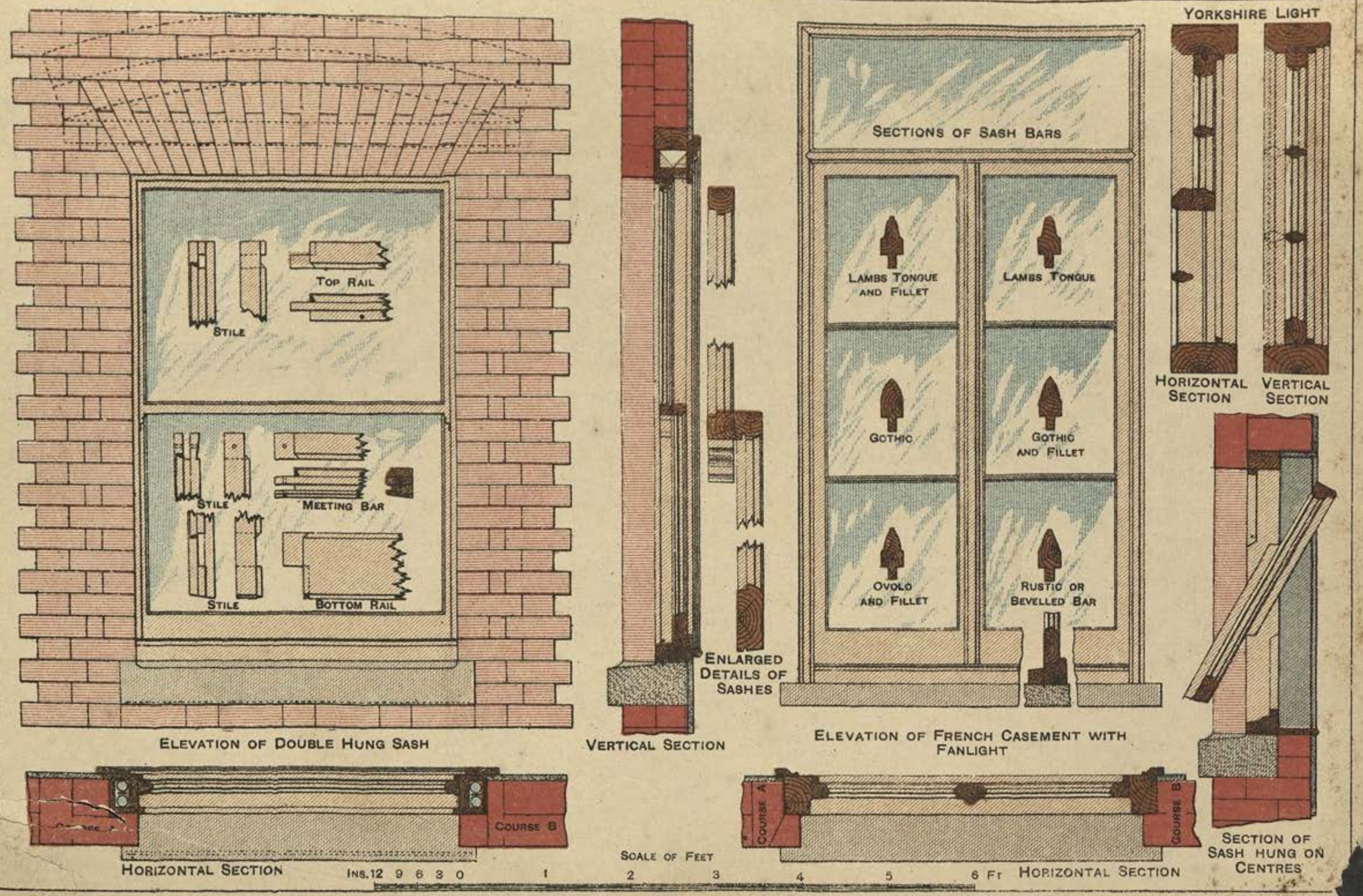


Fig. 1107.—Setting Out Parabolic Curve.

L = span in feet.
w = load in tons per foot run.
D = depth in feet.
s = intensity of stress in tons per square inch, = 5 tons tension, 4 tons compression.
M = bending moment in ton-ft.
A = sectional area of flange in square inches.
S = total stress on each flange in tons, in tension or compression.


Yorkshire Light
Horizontal Section
Vertical Section
Top Rail
Stile
Stile
Meeting Bar
Stile
Bottom Rail
Elevation of Double Hung Sash
Vertical Section
Enlarged Details of Sashes
Sections of Sash Bars
Lambs Tongue and Fillet
Lambs Tongue
Gothic
Gothic and Fillet
Ovolo and Fillet
Rustic or Bevelled Bar
Elevation of French Casement with Fanlight
Section of Sash Hung on Centres
Course B
Horizontal Section
Course A
Course B
Horizontal Section
Scale of Feet
Ins. 12 9 6 3 0 1 2 3 4 5 6 Ft

WINDOW SASHES

$$M = \frac{wL^2}{8},\ S = \frac{M}{D},\ A = \frac{S}{s}$$

$$M = \frac{2 \times 30^2}{8} = 225 \text{ ton-ft.}$$

$$S = \frac{225}{2{\cdot}5} = 90 \text{ tons.}$$

$$A = \frac{90}{4} = 22{\cdot}5 \text{ sq. in., compression flange.}$$

$$A = \frac{90}{5} = 18 \text{ sq. in. net, tension flange.}$$

Top flange compression, gross area = 22·5 sq. in., say 15 × 1½. Bottom flange, tension, net area = 18 sq. in., say 15 × 1½ – four ¾-rivet holes = 12 × 1½. The required girder is 2 ft. 6 in. high; top and bottom formed with three ½-in. plates riveted to the ⅜-in. web with 3 in. × 3 in. × ⅜ in. angles; stiffeners at 4-ft. intervals. In practice the span is taken from centre to centre of bearings, the depth from centre to centre of flanges, the load over the full length of girder, and the weight of the girder is allowed for.

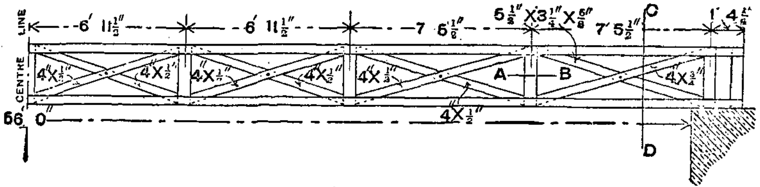


Fig. 1108.—Lattice Girder.

Determining Length of Flange Plates.

Fig. 1107 shows the best method of setting out a parabolic curve to determine where the flange plates terminate in the boom of a girder. The plates are drawn to the same scale as the elevation of girder for their length, and full size for the thickness, the area of one flange of the angle irons being added as if it were a thin inner plate. The length of the parabola is from centre to centre of the bearing surfaces, and the height is equal to the calculated sectional area divided by the width of the flange, so that it is often a trifle below the top edge of the plates. To set off the parabola, take twice the calculated height on the centre line and join the extremities. Divide the sides of the triangle into equal parts, and number them 1, 2, etc., as shown. Now join 1′ 1, 2′ 2, etc., and without a curve having been actually drawn, the underside of these lines will appear to be a parabolic curve, and may be inked in with a French curve. The various plates should be continued beyond the curve to a distance equal to half the length of a cover plate, although some designers only carry them one rivet hole beyond. With a concentrated load the stresses vary as ordinates to a triangle, and therefore, instead of a parabola a triangle would be drawn to give the terminations of the plates. This is the simplest method of working to get a true result. An approximate result may be obtained by making the inner plate the full length, and the outer one, when there are two, two-thirds to three-fourths of the length of the inner one. If there are three plates, the middle one would be about 0·85, and the outer one 0·65 of the inner one. With four plates they would be respectively about 1·00, 0·90, 0·75, 0·55.

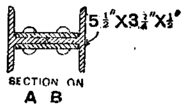


Fig. 1109.—Section of Vertical Part of Lattice Girder.

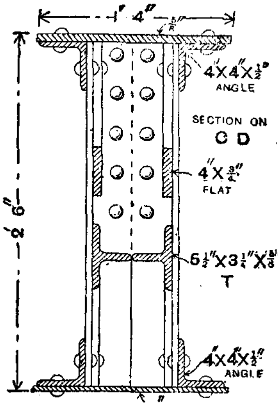


Fig. 1110.—Cross Section of Lattice Girder.

Strength of Lattice Girder.

It is required to know the safe distributed load for the lattice girder shown by Fig. 1108.

Effective span = say 58 ft. Bending moment $= \frac{WL}{8} = \frac{W \times 58}{8} = 7{\cdot}25\ W$. Mean depth = say 2 ft. 4½ in. = 2·375. Stress in flange at centre $= \frac{7{\cdot}25\ W}{2{\cdot}375} = 3{\cdot}053\ W$. Net sectional

area of bottom flange, including angle irons = say 11 sq. in. Allowance for tension = 5 tons per square inch. Resistance of bottom flange = 11 × 5 = 55 tons. Then stress 3·053 W = resistance 55; $\therefore$ W $= \frac{55}{3·053} = 18$ tons safe load distributed, assuming the bottom flange to be the weakest part, as it appears to be.

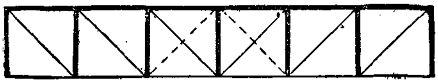


Fig. 1111.—Pratt Truss.

Sections of the girder on A B and C D respectively are represented by Figs. 1109 and 1110.

Warren Girders.

Warren girders were at one time much used on the Indian state railways and elsewhere, but have not been generally approved, because a single defective rivet might endanger the whole structure, although theoretically these girders are a very efficient form. The sections of the different parts are exactly similar to any other braced girder; namely, flat bars for tension members, and angles or tees for compression members, and the joints are also formed in a similar manner. A lattice girder is virtually a double Warren girder, and a failure of one bar would simply throw a greater stress on the remainder; the introduction of verticals furnishes a still further safeguard.

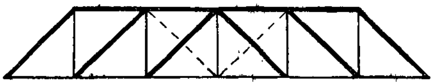


Fig. 1112.—Howe Truss.

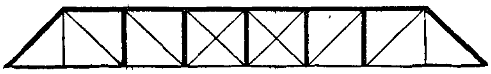


Fig. 1113.—Modified Pratt Truss.

Trussed Girders.

The difference between a Pratt truss and a Howe truss is shown in Figs. 1111 and 1112. In the Pratt truss (Fig. 1111) the compression members are vertical and therefore at their minimum length. In the Howe truss (Fig. 1112) the diagonals are in compression. It is a simple matter to find the stresses, by calculation or otherwise, for a trussed girder under a uniformly distributed dead load, but quite another matter when a live load travelling along from one end is concerned, and it is only the latter that is of any practical use for railway work. The depth is usually equal to the width of one bay, and the whole span divided into six to ten bays.

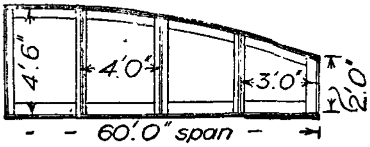


Fig. 1114.—Hog-backed Girder.

The Pratt truss is usually adopted for iron construction, and the Howe truss for timber. The former is sometimes made as shown in Fig. 1113, and might then be mistaken at first glance for the latter type. In Pratt truss girders of 143-ft. span for a highway bridge at Springville, New York, the length of each bay was 14 ft. 3⅝ in., and the depth of girders 18 ft., all centre to centre. There were in all ten bays, the central four being braced both ways to allow for the effect of varying load.

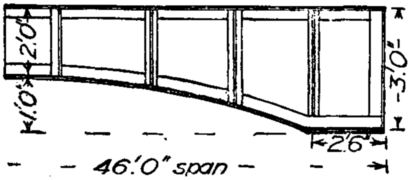


Fig. 1115.—Girder with Arched Soffit.

Counterbracing.

In a braced structure such as a lattice girder-bridge subject to a travelling load, the nature of the stress in some of the braces towards the centre will vary according to the position of the load. Although a member might be made strong enough to take either tension or compression, such a course is not always convenient, and the more usual plan is to counterbrace the

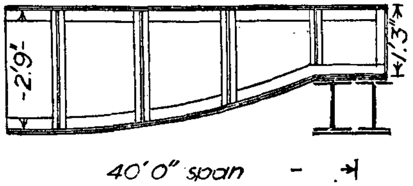


Fig. 1116.—Fish-bellied Girder.

girder towards the centre—that is, to add some extra braces in the opposite direction, which not only stiffen the girder, but provide for taking up the stress by tension instead of by compression. *Fig. 1111 shows a Pratt truss with* counterbracing, and Fig. 1112 a Howe truss. The counterbracing in each is dotted; the thick and thin lines show compression and tension respectively, when loaded uniformly over the whole span.

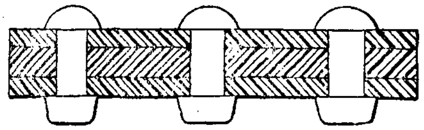
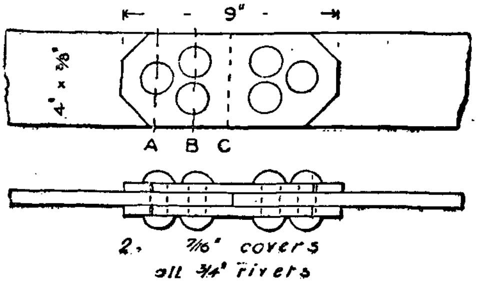


Fig. 1117.—Cross Section of Riveted Joint in *Tie-Bar.*

Hog-backed, Fish-bellied, and Other Girders.

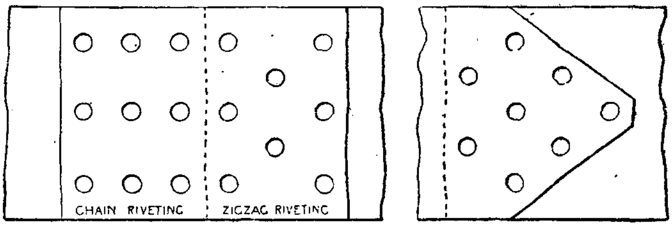
In all girders the bending moment depends only upon the method of loading and supporting, and the amount of load and width of span. The stress varies inversely as the depth, so that the bending moment at any section divided by the depth gives the stress in either flange at that section. Fig. 1114 shows what is called a hog-backed girder; its object is to simplify the construction by proportioning the depth to some extent to the bending moment, so that the stress along the flange will not vary so much as in a parallel girder. If the parabola of bending moment be drawn for the given span, then any ordinate divided by the depth at that point will give the flange stress. Fig. 1115 is a girder with arched soffit. It is the least economical form, unless the ends can be securely bolted horizontally to some fixed and rigid structure. The object is to give head room or to improve the appearance. The method of designing will be the same as in the previous case. Fig. 1116 is a fish-bellied girder. The object is the same as that of Fig. 1114, but adapted to the case of a central station traveller where the crab or winch can travel backwards and forwards along the *top flange.*



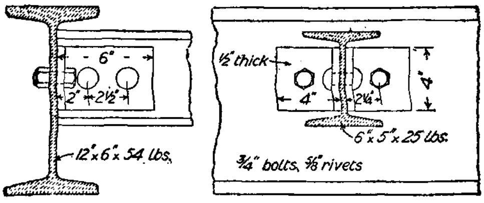
Figs. 1120 and 1121.—Double-cover Riveted Joint.

Riveted Joints.

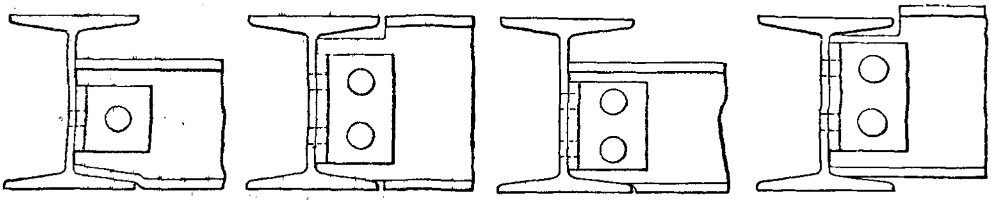
The cross section of a riveted joint in an iron tie-bar is presented by Fig. 1117. A plan of the joint, showing nine ¾-in. rivet holes arranged as chain riveting, and eight on the other side arranged as zigzag riveting, is given by Fig. 1118. The length allowed for zigzag riveting is insufficient, and the correct method is shown in Fig. 1119. Zigzag riveting is often the stronger, because if the cover plates be of the proper thickness, and the rivets of proper diameter, the effective breadth of the bar would be reduced by an amount equal to the diameter of one or two rivets only; whereas the chain riveting may often reduce the effective breadth by an amount equal to the combined diameters of three rivets. A double-cover riveted joint, as shown in Figs. 1120 and 1121, will be the best for joining two wrought-iron plates of the dimensions indicated. The full strength of



Figs. 1118 and 1119.—Chain and Zigzag Riveting.

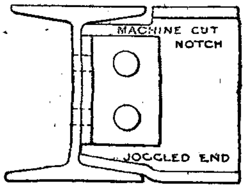


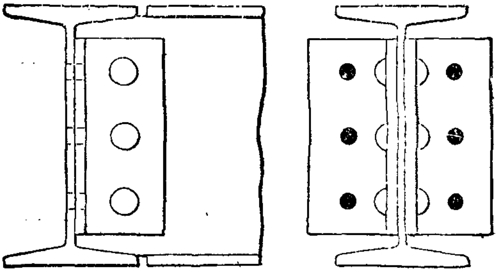
Figs. 1122 and 1123.—Rolled Joists connected with Angle Brackets.



Figs. 1124 to 1127.—Notching of Joists and connecting with Angle Brackets.

the plate will be 4 × $\frac{5}{8}$ × 22 = 55 tons. Each $\frac{3}{4}$-in. rivet would have an ultimate strength in double shear of 15 tons, or 40 tons per sq. in. bearing pressure. The resistance to fracture through the first rivet hole A would be (4 − $\frac{3}{4}$) × $\frac{5}{8}$ × 22 = 44·7 tons. The resistance to fracture through B and shear A would be (4 − 1½) × $\frac{5}{8}$ × 22 + 15 = 49·4 tons. The resistance to shear all the rivets on one side would be 15 × 3 = 45 tons. The resistance to tear the covers at C would be 2 (4 × $\frac{3}{8}$) × 22 = 66 tons, but this could not happen owing to the loss of section in the adjacent rivet holes making a weaker line. The resistance to tear the covers through B would be 2 × (4 − 1½) × $\frac{3}{8}$ × 22 = 41·25. The resistance to crush all the rivets in the holes on one side of joint would be 3 × $\frac{3}{4}$ × $\frac{5}{8}$ × 40 = 56·25 tons. The weakest part is therefore the tearing of the covers through B. This may be improved by increasing the thickness to $\frac{7}{16}$ in. each, when the strength will be brought up to 2 (4 − 1½) $\frac{7}{16}$ × 22 = 48·1 tons, leaving the weakest part now through the plate itself at the first rivet hole, and as the number of rivets here cannot be further reduced, this is the maximum strength, or an efficiency compared with the original strength of the plate of 81 per cent.





Figs. 1128 to 1130.—Notching of Joists and connecting with Angle Brackets.

Connecting Rolled Joists.

Standard angle bracket connections formed of plate bent to a right angle (not pieces of angle bar) are made for rolled joists. The brackets for a 9-in. rolled joist are 4 in. by 4 in. by ½ in. by 6 in. long, with two $\frac{3}{4}$-in. rivets in each, and two bolt holes in each for bolting up to the main girder with $\frac{3}{4}$-in. bolts. The load on the beam has practically nothing to do with these connections, as the small joists take their bearing on the larger ones, and the larger joists rest on good bearings. The angle brackets help to tie the rolled joists and therefore the building, and could not safely be omitted. Assume that a 6-in. by 5-in. steel joist is to be fixed on the web of a 12-in. by 6-in. joist with an angle bracket and bolt. Figs. 1122 and 1123 show the angle brackets that are necessary for making the connection, the brackets being riveted to the small joists and bolted to the large one. If the load that has to be carried by the small joists is 8 tons, a load of 4 tons has to be carried by the connection. The shear strength of steel rivets is 5 tons per square inch in single shear, and 7½ tons in double shears the difference between these figures and the ultimate strength of the material is to allow for contingencies. The rivets in the brackets will be in double shear, and as there are two rivets, each must resist 2 tons at the rate of 7½ tons per square inch sectional area, which will thus equal 7·5 = 0·27 sq. in. Reference to a table of areas will show that this resistance is given by $\frac{5}{8}$-in. rivets. The bolts will be in single shear, and will without doubt be of wrought iron, which will only take 4 tons per square inch. There are two bolts, so that each will have to carry 2 tons, and $\frac{2}{4}$ = 0·5 sq. in., which will be given by $\frac{13}{16}$ in. diameter, but $\frac{3}{4}$ in. diameter giving 0·44 sq. in. would probably be adopted. Different methods of notching when connecting joists by means of angle bracket are illustrated by Figs. 1124 to 1128. Angle brackets for connecting girders of the same depth are shown by Figs. 1129 and 1130.

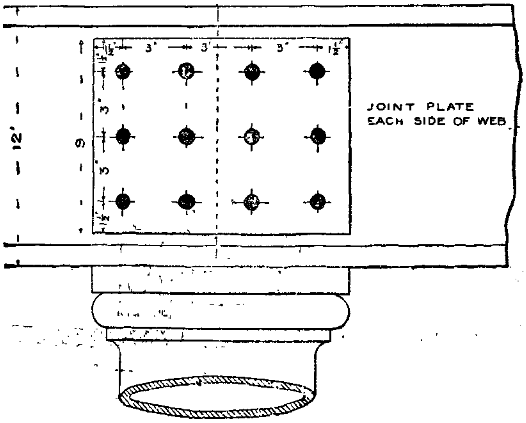
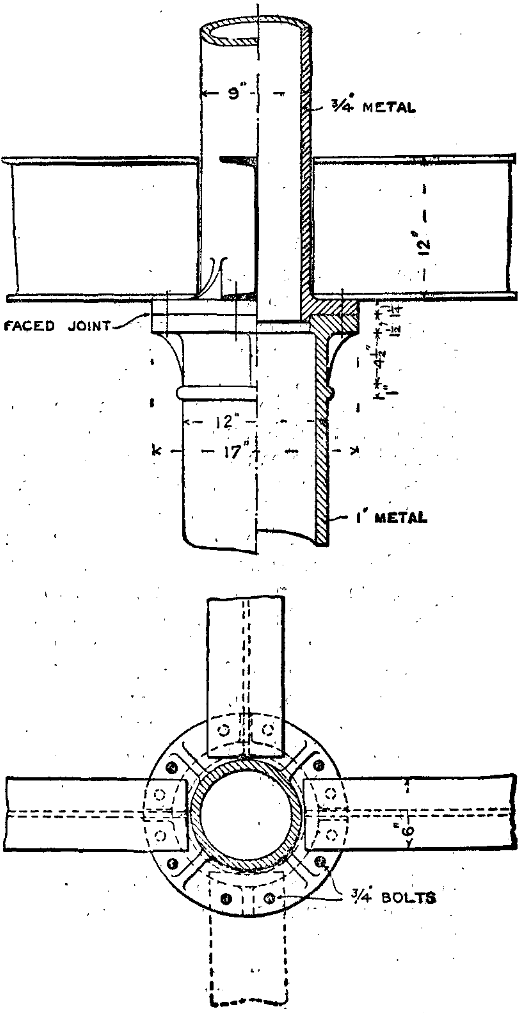


Fig. 1131.—Junction of Rolled Joists on Head of Column.

Connection of Joists at Head of Column.

The ordinary junction of rolled joists on the head of a column is illustrated by Fig. 1131. The head of a 12-in. cast-iron column for a warehouse floor can be connected in various ways to the main girder and the column above. Figs. 1132 and 1133 show four rolled joists meeting on the head of a 12-in. cast-iron column, with another column 9 in. in diameter above it. Making the joint below has the advantage of keeping the projecting flange clear for any desired arrangement of floor covering. Take another and a somewhat similar case. Figs. 1134 and 1135 show a cast-iron column, 7 in. diameter at the necking, which carries two lines of rolled girders at right angles to each other, the one 12 in. deep and the other 10 in. deep, and a similar column above. A specification to govern the supply of the columns in this second case may be as follows:—The columns to be shaped in accordance with the drawings and cast vertically from best cold blast grey metal of the second melting. The metal to be of uniform thickness all round, and free from blowholes, honeycomb, scabs, cold shuts or other defects. All re-entering angles to be filleted with curved fillets. The flanges to be square to the axis and with bolt-holes cored in $\frac{1}{8}$ in. larger in diameter than the bolts. The ends of the columns to be machine faced and jointed with red-lead to ensure perfect bearing. Test bars 2 in. by 1 in. to be cast from each melting, and to be tested as may be directed.



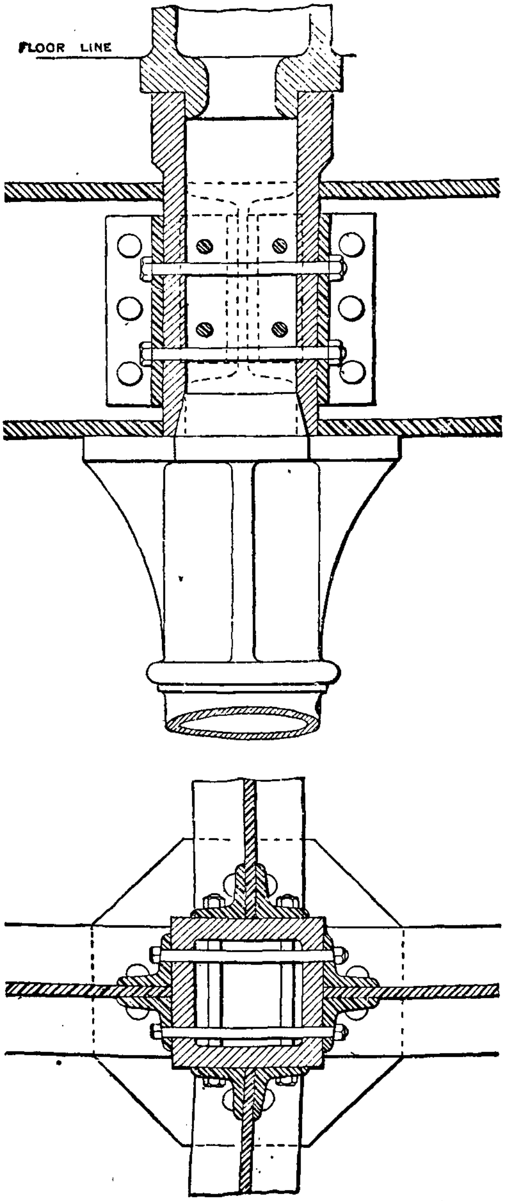
Figs. 1132 and 1133.—Connecting Head of Column to Main Girder and Column above.

Pitch of Girder Rivets.

The pitch of rivets in the flanges of a built-up girder may theoretically be increased towards the centre, but generally the more convenient usage is to keep the pitch uniform. The pitch must not be so great that the outer plate of the compression flange can spring between any two rivets and allow moisture to gain access to the joint. This limit is understood to be reached when the pitch is sixteen times the thickness of the plate; thus for ½-in. plate the maximum pitch is 8 in. Towards the ends of the girder much closer pitch than this is necessary, in order to allow for the horizontal shear.

Pressure on Bearing Area of Rivets.

The following describes the method of calculating the value of the crushing stress and pressure on the bearing area of rivets. As rivets that are properly driven entirely fill the rivet holes, the whole semi-circumference is in action at the same time on the side to which the rivet is pulled to cause a pressure on the bearing area; but as the circumference bears a constant ratio to the diameter, the latter is taken as the measure of the rivet, and the thickness of the plate as the other dimension, the diameter multiplied by the thickness giving the bearing area. At least two plates must be joined in order to obtain shear stress, and the shear stress is that which causes the pressure on the bearing area, so that with a lap joint of two ½-in. plates with ¾-in. rivets, the bearing area of each rivet will be ¾ × ½ = ·375 sq. in. With regard to the crushing action in the hole, Prof. Unwin says, "The value of the crushing stress which produces injury to the tenacity or shearing resistance of the joint is very uncertain. In the case of steel joints there is no indication of injury with crushing pressures of 50 tons per square inch. With the ordinary proportion of rivets, the crushing action need in no case be considered in single shear joints, and only in double shear joints when the plates are less than ¾ in. thick." The ultimate bearing pressure per square inch that will crush material may be taken as: single shear, iron 30 tons, steel 40 tons; double shear, iron 40 tons, steel 50 tons. The working pressure on the bearing area of rivets, bolts, and pins should not exceed 7½ tons per square inch in iron, and 10 tons in steel; but unless there is any special reason for permitting a maximum, the pressure will generally be limited to 5 tons and 7½ tons respectively. In American practice the bearing pressure is limited to 15,000 lb. per square



Figs. 1134 and 1135.—Cast-Iron Column carrying Two Lines of Rolled Girders.

inch, but Prof. Unwin is "inclined to believe that the importance of crushing action has been exaggerated."

Joint in Lattice Girder.

Fig. 1136 shows a joint in which a T-iron flange 3 in. by 4 in. by ⅜ in. is connected to a double-channel vertical and also to two flat tension bars, by means of two $\frac{5}{16}$-in. plates and ten ⅝-in. bolts, two to each of the flat bars. This joint was asked for in an examination question, but is a most unusual one, and rivets would invariably be used in preference to bolts.

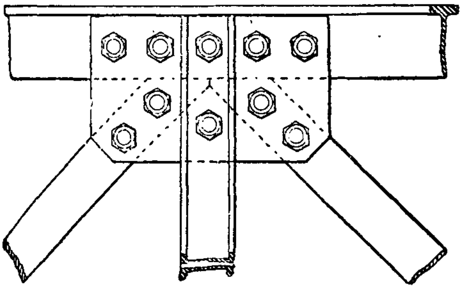


Fig. 1136.—**T-Iron Flange bolted to Double-Channel Vertical and Flat Bars.**

Making Rivet Holes in Girders.

In the construction of steel girders the rivet holes may be punched and afterwards drilled out ⅛ in. larger, or may be drilled the full size without any punching, but must not under any circumstances be punched the full size, as the metal will then be starred round the hole and seriously damaged. If reamered out instead of being drilled, after punching, the probability is that the holes would be punched too large, in order to save labour in reamering. No drifting is allowable in good work; the holes, being properly marked off from templates, should come fair in the different plates.

Cutting Rolled Joists.

If power is available, a wrought-iron disc, like a circular saw without teeth, and running at a very high velocity, say, 12,000 ft. per minute at the circumference with a jet of water playing on it, might be used Or holes may be drilled as close as possible along the line of cut, and a cross-cut chisel used to remove the intermediate portions. A Boyer power tool might be employed instead of the hand cross-cut, but this would require an air compressor and receiver for producing and regulating the air supply.

Girders to Support Floors.

Floor of 38-ft. Span.—A floor 38 ft. by 30 ft. is to be supported by a girder running the 38-ft. way. A single girder of 38-ft. span down the centre of the room would require to be of steel composed of a rolled joist 20 in. by 7½ in. by 89 lb., with two 12-in. by ⅝-in. plates on each flange, making the whole girder 22½ in. by 12 in. by 195 lb. per ft. The floor may then be carried by 11-in. by 3-in. fir joists resting on a 3½-in. by 3½-in. by ½-in. angle steel, riveted to web on each side, as shown in Fig. 1137.

16-ft. Span Floor loaded 1¼ Cwt. to the Foot.—A span of 16 ft. and distance of 10 ft. centre to centre, with a load of 1¼ cwt. to the ft. super., would give a total distributed load on each joist of $\frac{16 \times 10 \times 1\frac{1}{4}}{20} = 10$ tons, which will require a section equivalent to Dorman, Long & Co.'s *G 10a, 10-in. by 5-in. by 29-lb. rolled steel joist, or *G 10, 10-in. by 5-in. by 35-lb. rolled steel joist. The usual method of finding the section is to refer to the catalogue of a reliable manufacturer and choose a section that will carry the required load and have a depth in inches of not less than half the span in feet.

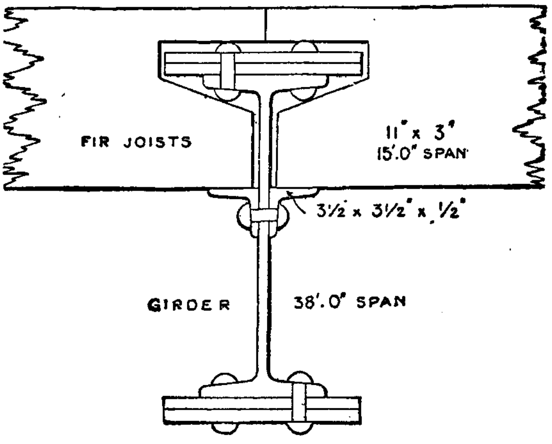
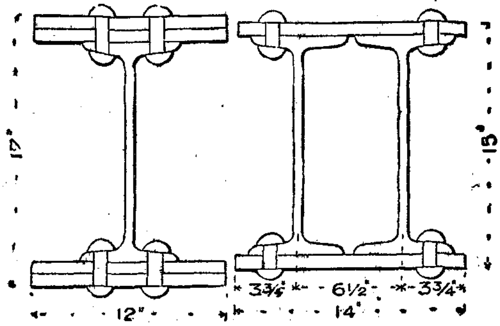


Fig. 1137.—Floor supported by Girder.

A roughly approximate result may be arrived at by the formula $w = \frac{\mathrm{W\,L}}{c}$, where w = pounds per foot run, W = tons distributed, L = span in feet, c = width of flange in inches. In the present case, as the load is fairly heavy, a rather deep joist, say 10 in., must be provided; then, taking the flange half the depth, say 5 in. wide, the result is $w = \frac{10 \times 16}{5} = 32$ lb. per foot run.



Figs. 1138 and 1139.—Sections of Joists.

24-ft. Span Warehouse Floor loaded 4 Cwt. to the Foot.—Rolled iron joists at 8 ft. centres are required to carry a warehouse floor. Taking the total load at 4 cwt. per ft. super., the span at 24 ft., and the depth of joists at 14 in., it is required to determine the amount of metal in the flanges in order that it may not be exposed to a greater stress than 5 tons to the inch. This example will be worked out as follows:—24 ft. span × 8 ft. centres × 4 cwt. per ft. super. = 768 cwt. total load on each joist. Stress on each flange = W L ÷ 8d = (768 cwt. × 24 ft. span × 12 in. in 1 ft.) ÷ (8 constant × 14 in. depth) = 1,975 cwt. Then 1,975 cwt. ÷ (5 tons × 20 cwt. in 1 ton) = 19·75 sq. in. This is clearly much more than one 14 × 6 rolled joist can furnish, and it will be necessary to provide for this area by adding plates. The flange of a 14 × 6 rolled joist is about ¾ in. average thickness, and the web ½ in. If single joists be adopted, the width of extra plates could not well exceed 12 in., and there would be at least two rivet-holes to deduct. Let the rivets be ⅞ in., with 4 in. pitch.

6 − (2 × ·875) = 4·25 net width joist
4·25 × ·875 = 3·72 sq. in. area.

Allow one-sixth of web = 12 × ·5 ÷ 6 = 1 sq. in.

19·75 − (3·72 + 1) = 15·03 sq. in. for plates
12 − (2 × ·875) = 10·25 net width of plates
15·03 ÷ 10·25 = 1·46 thickness of plates,

or say two ¾-in. plates, as in Fig. 1138; but this will be deficient in stability unless stiffeners are added. Two joists side by side with a plate or plates above and below may be adopted as an alternative, making what is known as a compound girder, as in Fig. 1139. Let the plates be 14 × ¾. Then the net area of tension flange, deducting two rivet-holes, will be—

14 − (2 × ·875) = 12·25 net width
12·25 × ·75 = 9·1875 sq. in. for plate
6 + 6 − (2 × ·875) = 10·25 net width
10·25 × ·875 = 8·96875 sq. in. for joists.

And allowing one-sixth of the depth of the webs as forming part of bottom flange, 12 × ·5 × 2 ÷ 6 = 2 sq. in. from webs, giving a total of 9·1875 + 8·96875 + 2 = say 20 sq. in., or just over the required amount.

Assembly-room Floor over Three Shops.—Three small shops in course of building are to have an assembly-room over them, the span between the walls being 17 ft. 7 in. For a clear span of 17 ft. 7 in. the least possible depth of steel joists for an assembly-room floor is 9 in., but a 10-in. by 4½-in. by 30-lb. or 10-in. by 5-in. by 29-lb. section would be much better. These joists may be placed 6 ft. apart, and it would be an advantage if 3½-in. by 1½-in. by 6-lb. joists were placed transversely every 6 ft. between the others, connected by angle brackets and carried by 2-in. by 2-in. by ¼-in. steel angle riveted to web of main girder joist (see Fig. 1140). The concrete filling (see Fig. 1140) should be the best Portland cement to 5 sea-beach gravel, and 6 in. thick. The centering should remain undisturbed for three weeks after the concrete is put in, and in the meantime there should be no traffic over it.

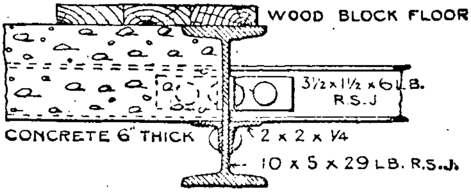
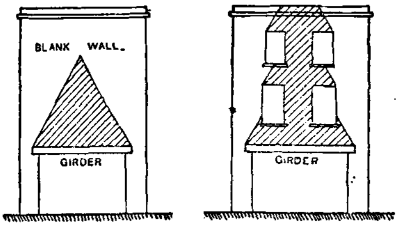


Fig. 1140.—Girder carrying Concrete and Wood Block Floor.

Girders over Shop Fronts.

Determining Load on Girder over Shop Front.—When no special reason exists for limiting the size of the girder, the whole of the brickwork above the girder, including the weight of the floors and the roof supported by the wall, may be provided for; but when the minimum size of girder must be provided, the shaded parts shown in Figs. 1141 and 1142, and drawn at an angle of 60°, may be taken as producing the active load, including, of course, any addition from the floors or the roof supported by the shaded part.



Figs. 1141 and 1142.—Determining Load on Girder over Shop Front.

Load of 100 Tons over Shop Front.—A girder is required to carry a weight of 100 tons over a shop front between its bearings. Assume the effective span to be 25 ft., mean depth about one-twelfth of the span = say 2 ft. The stress in the centre of the flanges $\frac{\text{W L}}{8\text{ D}} = \frac{100 \times 25}{8 \times 2} =$ 156·25 tons. Assume mild steel $6\frac{1}{2}$ tons per square inch net sectional area, then $\frac{156{\cdot}25}{6\frac{1}{2}} =$ 24 sq. in. sectional area required. Assume flanges 24 in. wide, then the thickness = 1 in., and the section is that shown in Fig. 1143, the flange of the angle bars making up for loss of material in the rivet holes. The weight of this girder would be $3\frac{1}{3}(2 \times 24 \times 1 + 2 \times 22 \times \frac{1}{2} + 4 \times 7\frac{3}{8} \times \frac{5}{8}) = 3\frac{1}{3}(48 + 22 + 18{\cdot}5) = 295$. Add 5 per cent. for rivets, 295 + 15 = 310 lb. per foot run. This is merely approximate, and in practice much more thought and care would have to be expended on the design, and any special conditions taken into consideration.

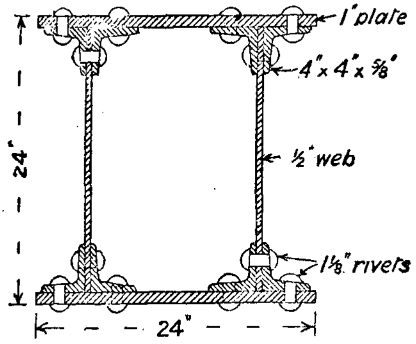


Fig. 1143.—Section of Box Girder.

Girders to Carry Hollow Wall.

It is supposed that two houses (now with open areas in front) are to be converted into two shops on the ground floor. The existing main outside wall is 12 in. hollow (two $4\frac{1}{2}$ in., tied together with the usual wall ties). The opening in the centre of the shop under the existing hollow front wall may be 10 ft. wide. As the gross load to be carried will be about 12 tons, two 6-in. by $4\frac{1}{2}$-in. by 20-lb. rolled steel joists will be required, bolted together through the webs with distance pieces, or two $6\frac{1}{4}$-in. by $3\frac{1}{2}$-in. by 18-lb., or two 7-in. by $3\frac{3}{4}$-in. by 18-lb., or two 8-in. by 4-in. by 19-lb., as may be preferred. They should have a bearing 18 in. long at each end, and will thus be 13 ft. over all. The piers under the ends should not be less than 18 in. by $13\frac{1}{2}$ in. built solid in cement brickwork, and bonded to existing walls. The section would be as shown in Fig. 1144. The soffit may be framed and panelled, or made in one piece out of American pine, or in cross pieces matched and beaded. The joist over the projecting shop front may be a single joist of the same section as the others, or 6-in. by 3-in. by 13-lb. would be sufficient. The latter size would also do for the rolled joist under the passage wall of the shop, or a 5-in. by 3-in. by 11-lb. would do equally well.

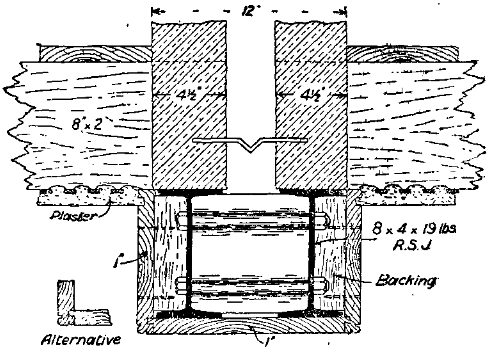


Fig. 1144.—Girders carrying Hollow Wall.

26-ft. Span Girder to Support Wall over Shop Front.

A stone building with ground and first floors, and eaves of the roof parallel with the front wall, is to have the lower part of the wall removed for the purpose of converting the building into a shop. The clear span will be 26 ft., which is considerable, and three rolled steel joists will be required to support the upper floor, wall, and roof. These joists should be not less than 14-in. by 6-in. by 46-lb. each, and would be better if connected by, say, four cast-iron distance pieces, each with two bolts, as in the section shown by Fig. 1145. If headway is of importance, three 12-in. by 6-in. by 54-lb. rolled steel joists may be substituted, but these joists will cost more owing to their extra weight. At least 12-in. length of bearing must be given at each end on a good squared stone template. The piers below should be rebuilt in cement unless they are very sound and substantial, or a cast-iron or rolled-steel stanchion, as shown in the illustration, may be put in. If the span were reduced to 25 ft., three 13-in. by 5-in. by 41½-lb. joists could be used, and the end piers would be safer. Another method would be to put intermediate supports, say one on each side of the central doorway. Solid cast-iron columns 4 in. in diameter, or, hollow, 4½ in. in diameter, with 1 in. thickness of metal, would do for this purpose if the columns had properly designed caps and bases, and the rolled joists might then be three, 8-in. by 4-in. by 19-lb., spaced to 6-in. centres.

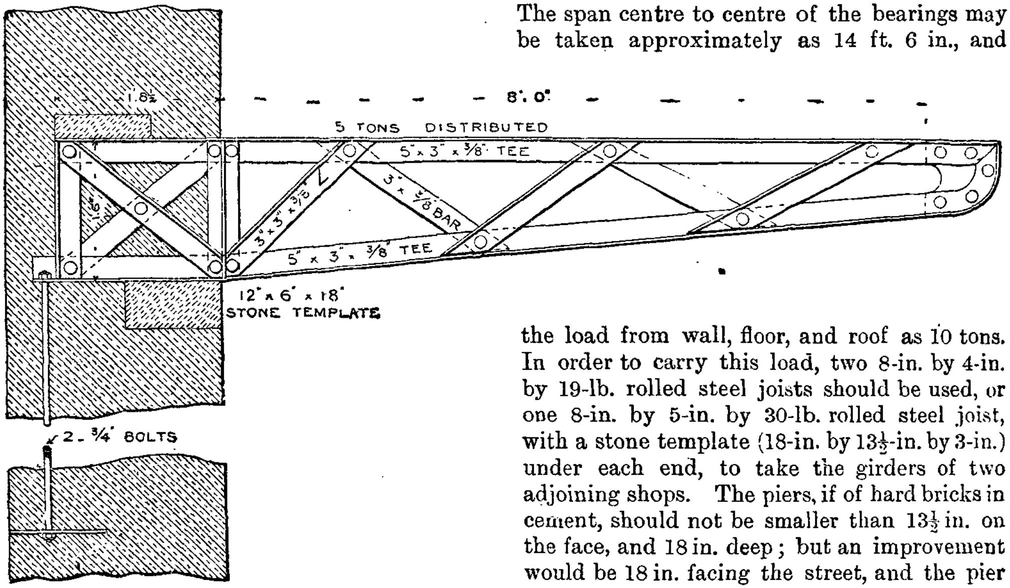


Fig. 1146.—Framed Steel Cantilever.

14-ft. 6-in. Span Girder to Support 10-Ton Load.—Assume that some houses to be erected will, in a few years' time, be turned into shops. The frontage of each house will be 15 ft. 3 in. It is desired to build in the necessary girders to support the brickwork of the floor above. The span centre to centre of the bearings may be taken approximately as 14 ft. 6 in., and the load from wall, floor, and roof as 10 tons. In order to carry this load, two 8-in. by 4-in. by 19-lb. rolled steel joists should be used, or one 8-in. by 5-in. by 30-lb. rolled steel joist, with a stone template (18-in. by 13½-in. by 3-in.) under each end, to take the girders of two adjoining shops. The piers, if of hard bricks in cement, should not be smaller than 13½ in. on the face, and 18 in. deep; but an improvement would be 18 in. facing the street, and the pier bonded into the party wall at the back. The girders would be, say, 15 ft. 1½ in. long, and the clear span 14 ft. 1½ in., giving a bearing of not less than 6 in. at each end, if the narrow way of pier faces the street, and a clear span of 13 ft. 9 in., with bearing of 8¼ in., if the pier stands the other way.

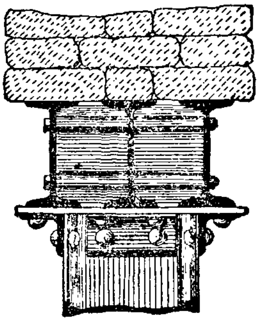


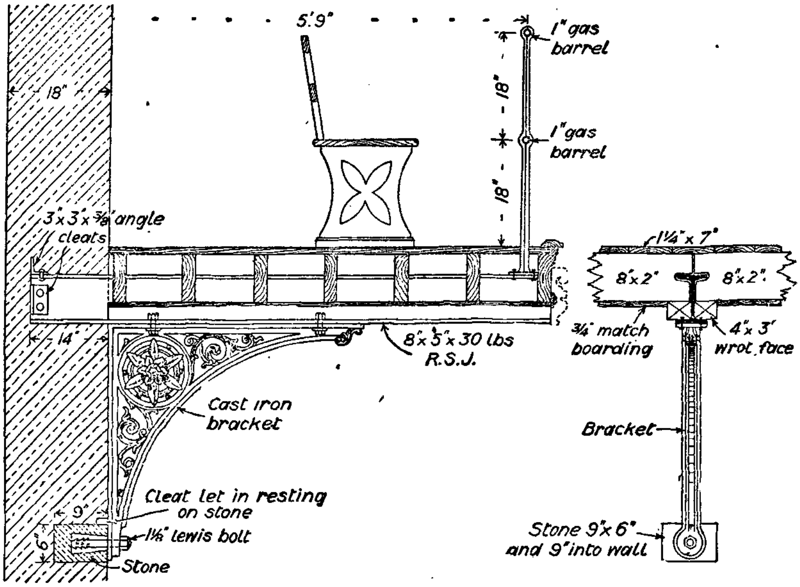
Fig. 1145.—Girders supporting Stone Wall over Shop Front.

Steel Cantilever.

The elevation of a framed steel cantilever 8 ft. from the wall is presented by Fig. 1146. The conditions are that the cantilever is not to be more than 1 ft. 6 in. deep, and must carry safely a distributed load of 5 tons. Allowing $\frac{1}{2}$ ton for the weight of the cantilever, the bending moment at the face of the wall = $5\frac{1}{2}$ tons total load × 4 ft. to the centre of gravity of the load = 22 ft. tons. Depth 1 ft. 6 in., then stress in flange = $\frac{22}{1\cdot5}$ = say 15 tons. Maximum tensile or compressive strength allowed = $6\frac{1}{2}$ tons per sq. in., then $\frac{15}{6\frac{1}{2}} = 2\cdot3$ sq. in. area. This will be given by a 4-in. by 3-in. by $\frac{3}{8}$-in. **T**, but preferably the **T** should be 5 in. by 3 in. by $\frac{3}{8}$ in., and the elevation will be as shown in Fig. 1146. Supposing the wall to be not less than three bricks thick, the cantilever will extend 1 ft. 10 in. into the wall; and the cantilever must either have above it 15 ft. of brickwork or must be anchored down say 8 ft. into the wall below by two $\frac{3}{4}$-in. bolts and a 12-in. by 18-in. by $\frac{1}{2}$-in. plate. A good York stone template 18 in. by 12 in. by 6 in. should be bedded under the cantilever at the inside face of the wall, and a 12-in. by 12-in. by 3-in. template on the top at the back end.

Cantilevers for Supporting Gallery.

Suppose that a gallery that will project 5 ft. 9 in. or 6 ft. from the wall, and will contain only one row of seats, has to be built round a large hall having 18-in. brick walls. Cast-iron columns cannot be used as supports for the gallery, because the floor space must be kept clear. It is proposed to rest the gallery on rolled steel joists, which are to project 5 ft. 9 in. from the face of the wall, these joists to be further supported by cast-iron cantilever brackets, etc. The minimum gross load that has to be allowed for will be 1 cwt. per ft. super., or for 5-ft. 9-in. span, say 6 cwt. per foot run of gallery. An 8-in. by 5-in. by 30-lb. rolled steel joist, 6-ft. span, supported at both ends, would carry 20 tons, but used as a cantilever would carry only one-fourth of this weight (say 5 tons); this is on the assumption that the tail end is securely bolted down; but if the walls are already existing, there is little probability of securing the ends sufficiently without additional support. For this reason, therefore, cast-iron brackets may have to be fixed below the cantilevers, but even then difficulty may be experienced in sufficiently tying back the rolled joists. A strap bolt from each joist carried through to the back of the wall, would make the best arrangement, but cleats (as



Figs. 1147 and 1148.—Cantilevers for supporting Gallery.

shown in Figs. 1147 and 1148) may have to do instead. In estimating the strength of the cantilevers, the better plan will be not to reckon on the support from the brackets ; then, at 6 cwt. per foot run of gallery, and a strength of 5 tons in the cantilever, the cantilevers must be placed not more than $\frac{5 \times 20}{6} = 16\frac{2}{3}$ ft. centre to centre, but may of course be closer. The cantilevers should preferably be placed in the solid piers between the windows, and assuming a centre distance of 12 ft., the fir joists between the cantilevers may be $\frac{12}{2} + 2 =$ 8 in. deep and 2 in. thick, the flooring $1\frac{1}{4}$ in. by 7 in., and the ceiling of $\frac{3}{4}$-in. matchboarding. The cast-iron brackets should be strong and ornamental, with lugs on each side at the top in order to take four $\frac{3}{4}$-in. bolts through the flange of the rolled joist. If the walls are sound and the mortar is good, and all new work is cut and pinned in cement, the proposed design, as shown in Figs. 1147 and 1148, may be considered a safe one.

Definition of Length of Column.

The length of a column, pillar, or strut of any kind in a formula is the unsupported length. If secured in two directions at right angles to each other at each floor, the length will be the height from floor to floor. In the case of a braced tower, the whole structure is taken as a single column free at the top, and each part is also taken as a column according to its unsupported length.

Meaning of Term "Crippling Load."

A crippling load on a column is one that produces, or is likely to produce, the first symptoms of failure. It is what might be called the practical breaking stress, as against the ultimate breaking stress which would be given by the absolute crushing and collapsing load.

Strength of Columns and Stanchions.

A truth that cannot be too often emphasised is that the strength of a column or stanchion depends upon so many things that no rough and ready rule, that would apply to all cases without calculation, can be given. However, if no objection can be urged against providing extra material in order to cover risk, and the load is estimated with an allowance for contingencies, the approximate rules given below may be taken. Cast-iron stanchions and hollow columns, safe load $1\frac{1}{2}$ tons per sq. in. sectional area; built-up steel stanchions, safe load $3\frac{1}{2}$ tons per sq. in. sectional area. These are only rough rules for those who cannot work out a mathematical formula and are content to waste material or to run the risk of failure.

Tables of Safe Loads on Columns and Stanchions.—As has been already suggested, the simpler formulæ for determining the strength of stanchions are only approximately correct and the more nearly anyone desires to ascertain the actual strength of a stanchion the more complicated are the formulæ that must be used. The following table gives a simple approximation to the strength of H stanchions in cast iron, wrought iron, and mild steel :—

SAFE LOAD ON STANCHIONS IN TONS PER SQ. IN.

Diameters in Length.	Cast Iron.	Wrought Iron.	Mild Steel.
8	5	4	5
10	4	$3\frac{3}{4}$	$4\frac{3}{4}$
13	3	$3\frac{5}{8}$	$4\frac{1}{2}$
15	$2\frac{1}{2}$	$3\frac{1}{2}$	$4\frac{3}{8}$
18	2	$3\frac{3}{8}$	$4\frac{1}{8}$
20	$1\frac{7}{8}$	$3\frac{1}{4}$	4
24	$1\frac{1}{2}$	3	$3\frac{3}{4}$
30	$1\frac{1}{8}$	$2\frac{5}{8}$	$3\frac{1}{4}$
32	1	$2\frac{1}{2}$	3
40	$0\frac{3}{4}$	2	$2\frac{1}{2}$
50	$0\frac{1}{2}$	$1\frac{3}{8}$	$1\frac{3}{4}$
60	$0\frac{1}{4}$	$0\frac{3}{4}$	1

The diameter in the above table is the least width of the stanchion, being presumably the direction of failure. For cast-iron hollow columns, with a thickness of metal equal to one-twelfth the diameter, the following may be taken :—

SAFE LOAD ON CAST-IRON COLUMNS.

Up to 10 diameters long = 5 tons per sq. in.
10 to 15 „ „ = 4 „ „
15 to 20 „ „ = 3 „ „
20 to 25 „ „ = 2 „ „
25 to 30 „ „ = $1\frac{1}{2}$ „ „
30 to 35 „ „ = $\frac{3}{4}$ „ „

To allow for contingencies $\frac{1}{64}$ in. should be added to the thickness for every $\frac{1}{8}$ in. the thickness falls short of 1 in. Columns often have to resist side blows. Another and very similar table is :—

Up to 8 diameters long = 5 tons per sq. in.
„ 10 „ „ = 4 „ „
„ 13 „ „ = 3 „ „
„ 15 „ „ = $2\frac{1}{2}$ „ „
„ 18 „ „ = 2 „ „
„ 24 „ „ = $1\frac{1}{2}$ „ „
„ 32 „ „ = 1 „ „
„ 40 „ „ = $\frac{3}{4}$ „ „
„ 50 „ „ = $\frac{1}{2}$ „ „

Safe Load carried by Cast-Iron Stanchion 12 ft. High.—It is required to know the safe load that can be carried by a cast-iron stanchion 12 ft. high and of the section shown in Fig. 1149. In finding the strength of a stanchion of this section, there will be two steps : first, to find the strength of the web between two stiffeners ; and secondly, to find the strength of the stanchion as a whole. The approximate safe load on a cast-iron stanchion may be taken as in the last table given above. In the present case the web is $1\frac{1}{4}$ in. thick, which is equivalent to least diameter, and the unsupported height 3 ft. = 36 in. ; the ratio of length to diameter is thus $\frac{36}{1\frac{1}{4}} = 28\cdot8$. Taking the next higher ratio in the table, the safe load would be 1 ton per square inch. Now taking the whole column, the least diameter is 6 in. and the length 12 ft., giving a ratio of $\frac{12 \times 12}{6} = 24$. This by the table above would permit a safe load of $1\frac{1}{2}$ tons per square inch, but owing to the greatly projecting webs it would be wise to limit it to the previous figure of 1 ton per square inch. The area of the section is $6 \times 1\frac{1}{4} + 2\,(9 - 1\frac{1}{4}) \times 1\frac{1}{4} = 7\cdot5 + 19\cdot4 = 26\cdot9$, say 27 sq. in., and therefore a safe load of 27 tons may be put upon it.

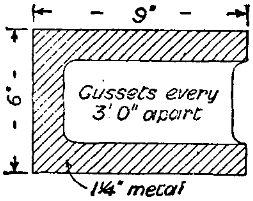


Fig. 1149.—Section of Cast-Iron Stanchion.

Relative Strengths of Stanchions in Varying Conditions.—The relative strength of stanchions with (*a*) both ends pivoted, (*b*) one end pivoted and one end fixed, (*c*) both ends fixed, is given by various authorities as follows :

	a	*b*	*c*
Stoney, Hodgkinson, Parkinson	1	2	3
Unwin, Merriman ...	1	2	4
Moos	1	1·77	4
Gordon	1	$2\frac{1}{3}$	4
Crehore (theoretically) ...	1	2·28	4
Crehore (practically) ...	1	2·25	4
Molesworth	1	2	8

The writer is inclined to agree with the first statement above, being the number of points at which fracture must necessarily occur, as one may reasonably suppose that if *x* tons will break a stanchion in one place, 2*x* tons will be required to break the stanchion in two places at the same time, and so on. The approximate safe load in tons per square inch for a stanchion of rolled joist or built-up section from fifteen to forty-five diameters long is $5\cdot5 - 0\cdot075\,\frac{l}{d}$, l and d being the length and least diameter in the same units, either feet or inches.

Gordon's Formula for Calculating Built-up Stanchions.—Many different formulæ have been devised for calculating built-up stanchions. Gordon's formula is one of the best known, but the coefficient is varied by different authorities. The following is a full statement of the formula with average coefficients :—l = length in inches ; d = least diameter in inches ; f = greatest intensity of stress in tons per square inch due to thrust and flexure when on the point of buckling ; for wrought iron = 18 ; for mild steel = 26 ; t = average thrust in tons per square inch on section, which will be the crippling stress per square inch when f is taken as above ; a = constant depending on condition of ends ; for both ends fixed = 1 ; for both ends pivoted = 4 ; for one end fixed, and the other pivoted = 2·5 ; for one end fixed, the other free = 16 ; c = constant depending on the shape of the cross section, and the nature of the material ; for rectangular or cylindrical solid bars of wrought iron or mild steel = 2,500 ; for cylindrical tubes of wrought iron or mild steel = 3,500 ; for angle, tee, cross, channel, rolled joist, bars and distance pieces, hollow, square, and built sections generally = 900.

$$t = \frac{f}{1 + \left(\frac{a\,l^2}{c\,d^2}\right)}$$

The proper factor of safety will be 4 + (·05 length ÷ least diameter).

Euler's Formula for Calculating Built-up Stanchions.—Results obtained with Gordon's formula may be checked by one founded on Euler—namely,

$$\text{W} = \frac{\pi^2}{m^2} \times \frac{n\,\text{EI}}{l^2}$$

Where W = working load in pounds, π = 3·1416, m = constant depending upon mode in which ends are fixed (say ·5 for column fixed both ends), n = factor of safety (say one-sixth), E = modulus of direct elasticity of material (say 29,000,000 lb. for mild steel), I = moment of inertia of section (see Molesworth's Pocket Book), l = length in inches. Built-up stanchions are somewhat troublesome to calculate, but a suitable section is generally obtained at about the third trial.

Fidler's Formulæ for Strength of Stanchions.—The following formulæ by T. Claxton Fidler are given as the most reliable known at the present time for strength of stanchions of given section :—

p = load in lb. to produce stress f.

f = ultimate compressive stress in lb. per sq. in.

R = resilient force of ideal column in lb. per sq. in.

L = length of column in inches. For fixed ends $l = \frac{6}{10}\text{L}$.

r = radius of gyration in inches measured in plane of easiest flexure.

E = modulus of direct elasticity of material.

Maximum p or breaking weight of ideal column

$$= \text{R} = \text{E}\pi^2 \times \left(\frac{r}{l}\right)^2.$$

Minimum p or breaking weight of practical

$$\text{column} = \frac{f + \text{R} - \sqrt{(f + \text{R})^2 - 2{\cdot}4 f\text{R}}}{1{\cdot}2}$$

E = 26,000,000 lb. for wrought iron.
14,000,000 lb. for cast iron.
29,000,000 lb. for mild steel.

f = wrought iron 36,000 lb.
cast iron 80,000 lb.
hard steel 70,000 lb.
mild steel 48,000 lb.
structural steel, average 60,000 lb.

$$\text{Factor of safety} = 4 + {\cdot}05\,\frac{l}{d}.$$

Calculating Solid Steel Round Columns.—The usual constant for solid bars is 2,500, but in his own practice the writer adopts 1,000. The difference in the various methods may be shown as below. Take, as an example, a 3-in. steel column 12 ft. high, having one end fixed and the other end rounded or imperfectly fixed. (*a*) By one published table 7·07 sq. in. area × 27 tons per square inch ultimate crushing strength ÷ 4 for factor of safety = 47·72 tons safe load. (*b*) By Gordon's formula and Shaler Smith's factor of safety

$$p = \frac{f}{1 + \frac{a}{c} \cdot \frac{l^2}{d^2}} = \frac{26}{1 + \frac{2{\cdot}5}{2500} \times \frac{(12 \times 12)^2}{3^2}} = \frac{26}{1 + 2{\cdot}3}$$

= 7·88 tons per square inch crippling stress.

$$\text{Then}\ \frac{7{\cdot}07\text{ sq. in.} \times 7{\cdot}88\text{ tons}}{6{\cdot}5\text{ factor safety}} = 8{\cdot}6\text{ tons safe load.}$$

(*c*) By the same formula, but with coefficient of 1,000 instead of 2,500 and 6 factor of safety, p = 4 tons, then $\frac{7{\cdot}07 \times 4}{6} = 4{\cdot}7$ tons safe load. The column would of course carry a heavier load if both ends were securely fixed.

Safe Load on Cast-Iron Hollow Column.

Gordon's formula for cast-iron hollow columns fixed at both ends is P = crushing load in pounds, f = 80000, S = sectional area in square inches, $a = \frac{1}{800}$, l = length in inches, d = least external diameter in inches. Then $\text{P} = \frac{f\,\text{S}}{1 + a\left(\frac{l}{d}\right)^2}$, which for a column 10 ft. high and 6 in. in diameter with ½ in. thickness of metal $= \frac{80000 \times 8{\cdot}6394}{1 + \frac{1}{800}\left(\frac{120}{6}\right)^2} = \frac{691152}{1{\cdot}5} = 460768$ lb. = 205·7 tons breaking weight. A factor of safety of not less than 6 should be allowed, which should be increased to 8 with a live load, or 10 if subject to shocks. Safe load say 20 tons.

Rolled Steel Joists Used as Stanchions.—Gordon's formula for rolled steel joists used as stanchions is $\text{P} = \frac{f\,\text{S}}{1 + a\left(\frac{l}{d}\right)^2}$ as before, but f = 50000 lb., and $a = \frac{1}{36000}$, according to the Cambria Steel Co., or f = 28 tons and $a = \frac{1}{900}$, according to other authorities. For a mild steel stanchion of 8-in. by 6-in. by 35-lb. section 10 ft. high, the breaking weight would be $\dfrac{50000 \times 10{\cdot}3}{1 + \dfrac{1}{36000}\left(\dfrac{120}{6}\right)^2} = 509340$ lb. $= 227{\cdot}4$ tons, or $\dfrac{28 \times 10{\cdot}3}{1 + \dfrac{1}{900}\left(\dfrac{120}{6}\right)^2} = 199{\cdot}7$, say 200 tons; the latter, being the lesser weight, is the safer, and allowing a factor of safety of 10, as before, the safe load would be 20 tons. Mild steel is, however, not subject to such contingencies as cast iron, and with a dead load acting vertically down the centre line, a factor of safety of 5 would be sufficient, making the working load 40 tons.

Strength of Twin Stanchion.—A stanchion formed of two uprights which are connected at intervals by webs is only of the same strength as two independent stanchions, unless the webbed stanchion is specially designed in order to utilise economically this method of construction. Assume the case of two solid cast-iron stanchions (each 5 in. by 2½ in.) 11 ft. 6 in. high. The formula is: $\text{W} = \dfrac{36s}{1 + \dfrac{3}{800}\left(\dfrac{l}{d}\right)^2}$, where W = crushing weight in tons, s = sectional area in sq. in., d = diameter or least width in inches, l = length in inches, factor of safety 6. Then safe load in tons on the double stanchion, $\dfrac{3}{800}$ for solid section =

$$2 \times \tfrac{1}{6} \times \frac{36 \times 12{\cdot}5}{1 + \dfrac{3}{800}\left(\dfrac{138}{2{\cdot}5}\right)^2} = \tfrac{1}{3} \times \frac{380}{1 + 11{\cdot}43} = 10 \text{ tons},$$

assuming that all internal angles are filleted, that the top and bottom have properly designed brackets, that the ends are fixed, and that the load passes through the axis.

Cast-Iron Column for carrying Centre of Floor.

It is required to find the shape and section of a cast-iron column that has to take the superincumbent weight in the centre of a basement measuring 40 ft. square, the floor above having to carry a weight of 4 cwt. per ft. super. The writer does not know of any formula that is applicable to this case for finding the load on a column; but probably the total would be somewhere between the two extremes of load on half span in each direction, and load on half area, that is, between 20 × 20 × 4 = 1600 cwt. = 80 tons, and $\dfrac{40 \times 4}{2} \times 4$ = 3200 cwt. = 160 tons. As there is only one column, and cast iron is cheap, the latter figure should be assumed to be correct. The height of the column would probably not exceed 10 ft., and the diameter would not be much less than 12 in. For a cast-iron column not exceeding ten diameters long the approximate safe load per square inch is 5 tons, requiring $\dfrac{160}{5} = 32$ sq. in. For a thickness of $\dfrac{1}{12}$ diameter the sectional area $= 0{\cdot}24d^2$, then if $32 = 0{\cdot}24d^2$, $d = \sqrt{\dfrac{32}{0{\cdot}24}} = 11{\cdot}67$, say 12 in., outside diameter, and 1 in. thickness. The strength of the assumed column should now be obtained by Gordon's formula, $\text{P} = \dfrac{f\,\text{S}}{1 + a\,\dfrac{l^2}{h^2}}$ where P = crushing load of pillar in pounds, f = 80,000 (ultimate resistance to crushing of a short block in pounds per square inch), or about 36 tons, S = sectional area in square inches, $a = \dfrac{1}{800}$ constant deduced from Hodgkinson's experiments, l = length, and h = least diameter both in same units of measure. Then the crushing load on the column $= \dfrac{36 \times 35{\cdot}5}{1 + \dfrac{1}{800} \times \dfrac{10^2}{1^2}} = \dfrac{1278}{1{\cdot}125} = 1136$ tons, and the maximum possible load being 160 tons, the factor of safety will be 7·1, which would be reasonable under the circumstances, but rather lower than the usual practice. If preferred, a cross-shaped section might be adopted, say about 15 in. by 15 in. by 1⅛ in., but the hollow circular section would generally be more suitable.

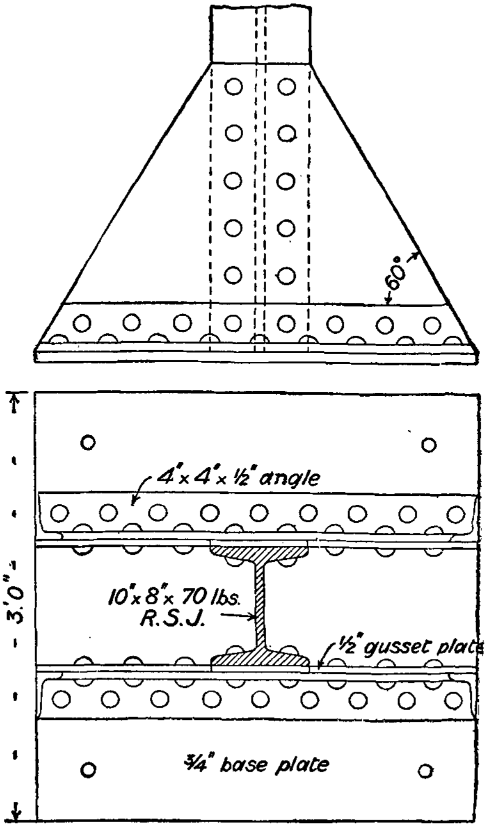
Effect of Taper on Strength of Column.

The effect of taper on a cast-iron column is to improve the appearance, and to give greater sectional area where the greater load comes, that is, towards the bottom, but the increase of section is often considerably beyond that which is requisite. In calculating the strength of a tapered column there is no strict theory to go on, but a reasonable course would be to take the mean diameter for the ratio of length to diameter, and the minimum diameter when calculating the sectional area. A taper column would thus have more strength than a cylindrical column of the same minimum diameter, and less than a cylindrical column of the same maximum diameter. Solid drawn steel columns would only be used for a large ratio of length to diameter where cast-iron columns would not be suitable.

Foundation for Stanchion.

Assume a rolled steel joist (section I and size 12 in. by 5 in.) weighing 32 lb. per foot to be used as a stanchion having a stationary load of 27 tons; the size of the required base is to be calculated, the ground being formed of compact earth. The size of the bottom flange will depend on the stone used as a support; say hard York stone or its equivalent, safe load 12 tons per ft. super., $27 \div 12 = 2{\cdot}25$ sq. ft. $\sqrt{2{\cdot}25} = 1{\cdot}5$ ft. square. The size of the stone depends on the strength of the supporting brickwork; say stock bricks in mortar, safe load 4 tons per ft. super., $27 \div 4 =$ say 7 sq. ft., $\sqrt{7} = 2{\cdot}65$, say 2 ft. 9 in. square. The size at the base of the brick footings depends upon the strength of the concrete; say lime concrete safe load 2 tons per ft. super. $27 \div 2 = 13{\cdot}5$, $\sqrt{13{\cdot}5} = 3{\cdot}7$ say 3 ft. 9 in. square, which is a brick measurement. The width of the concrete depends upon the safe pressure upon the earth below; say common compact earth safe load = 1 ton per ft. super. ∴ 27 ft. super. will be required, $\sqrt{27} = 5{\cdot}2$, say 5 ft. 3 in. square. The thickness should equal one and a half times the projection, therefore the thickness will be not less than 13½ in., say 1 ft. 3 in. If the soil is at all damp, the concrete should be made with Portland cement, and cement mortar should be used for the brickwork; nevertheless, owing to the wet soil, no reduction on account of the extra strength of cement over lime can be permitted in the size of the concrete.

Another Case.—The size of base plate for a rolled steel stanchion, 10 in. by 8 in., carrying a load of 130 tons, depends upon what it is standing on, whether hard stone, soft stone, brickwork, or concrete. Assuming it to stand on York stone capable of bearing a safe distributed load of 15 tons per ft. super., the area of the base plate should be $\frac{130}{15} = 8{\cdot}66$, say 9 sq. ft., or 3 ft. by 3 ft., with sufficient stiffening brackets, as in Figs. 1150 and 1151. If the York stone stands on stock brickwork in cement, the area of stone should be $\frac{130}{9} = 14{\cdot}44$ ft. super. $= 3{\cdot}8$ ft. width of side, or, adopting a brickwork dimension, say 3 ft. 9 in. The area of the bottom course of the footings, if standing on cement concrete, must be $\frac{130}{4} = 32{\cdot}5$ ft. super. $= 5{\cdot}7$ ft. width of side, or, adopting a brickwork dimension, say 5 ft. 7½ in., which will be given by five courses. The concrete should project not less than 6 in. all round beyond the footings, making it 6 ft. 7½ in. width of side, which will give about 3 tons per ft. super. on the soil. For a load of 130 tons a 10-in. by 8-in. by 70-lb. rolled steel joist should not exceed 6 ft. or 8 ft. high. Base plates are often made too small owing to false ideas of the strength of stone.



Figs. 1150 and 1151.—Base of Rolled Steel Stanchion.

Holding-down Bolts for Columns, Inclined Joists, etc.

Holding-down bolts to columns are generally short lewis bolts 6 in. to 9 in. long, and are only intended to prevent the shifting of the base. With a direct load, nothing more than this is necessary. When, however, the columns have to support an open shed roof in an exposed situation, or an overhead traveller or anything liable to side strain, the holding-down bolts should be proportioned to the possible strain. The calculation for strength required will be—length of column in feet multiplied by horizontal force at top in tons and divided by (say) distance apart of holding-down bolts in feet. This will give the upward pull in tons if the connection between bottom flange and column is sufficient to bear it. A 1-in. bolt is good for 1¼ tons working load, a 1¼-in. bolt for 2½ tons, and a 1½-in. bolt for 3½ tons. Then the block of concrete or stone must be heavy enough not to be canted over at the extreme pull. There is no rule for the number or size of bolts in the base of a stanchion; it is entirely a matter of judgment. Theoretically, the extreme calculation would be to take the stanchion as a lever, and so proportion the holding down that the stanchion would be liable to fail as a cantilever at its lower end with the same force that would tear the bolts or overturn the base, but probably few ordinary foundations would resist more than 25 per cent. of such an overturning force. The effect of an inclined joist with vertical support at upper end (see Fig. 1152) would be to produce a horizontal thrust at bottom, and therefore direct shear upon the bolts, of an amount equal to half the load, if the connection at upper end were severed. The condition here is the same as for a ladder placed against the wall, but it will vary with the shape and fixing of the ends.

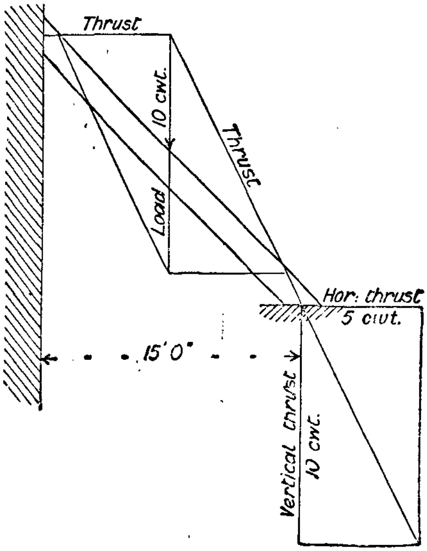
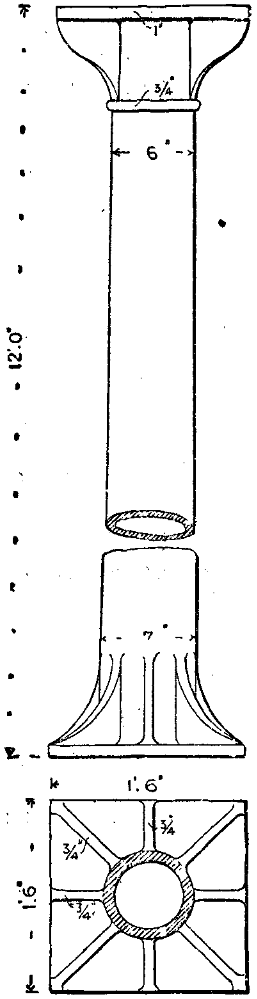


Fig. 1152.—Diagram showing Effect of Inclined Joists with Vertical Support at Upper End.



Figs. 1153 and 1154.—Cast-Iron Circular Column.

12-ft. Cast-Iron Circular Column to Support Load of 30 Tons.

A circular cast-iron column, 12 ft. high, capable of supporting a dead weight of 30 tons, is shown in elevation by Fig. 1153, and in sectional plan by Fig. 1154. The dimensions and thickness of metal are calculated as follows:—W = working load in tons; l = length in inches; d = diameter in inches; A = sectional area in square inches; c = constant = $\frac{1}{320}$ for columns fixed at one end; f = safe stress tons per square inch = 7 for cast iron; by Gordon's formula: $A = \frac{W\left(1 + c\frac{l^2}{d^2}\right)}{f}$. Say d = 6 in., then $A = \frac{30\left(1 + \frac{1}{320} \times \frac{144^2}{6^2}\right)}{7}$ = 12 sq. in.; 6 in. = 28·27 area, 28·27 − 12 = 16·27 area, 16·27 sq. in. corresponds to 4½ in. diameter, then $\frac{6 - 4\frac{1}{2}}{2} = \frac{3}{4}$ in. thickness. Say 6 in. diameter × ¾ in. thick, increasing the diameter 1 in. at the bottom for stability and appearance. (See Figs. 1153 and 1154.)

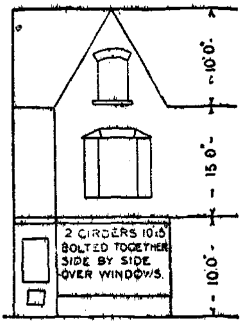


Fig. 1155.—Sketch Elevation of Shop Front.

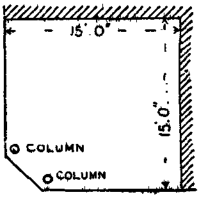


Fig. 1156.—Sketch Plan of Shop Front.

Iron Columns for Shop Front.

In the case shown in the rough sketch (Fig. 1155), each pair of girders will have a maximum of 25 tons to carry from all sources, for which 10-in. by 5-in. sections will be just about right. (Fig. 1156 is a sketch plan of the shop front.) Each column will have a

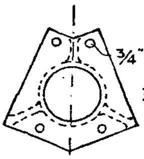


Fig. 1160.

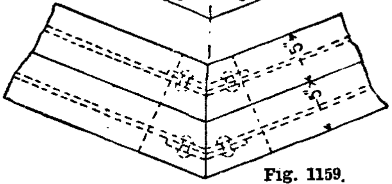


Fig. 1159.

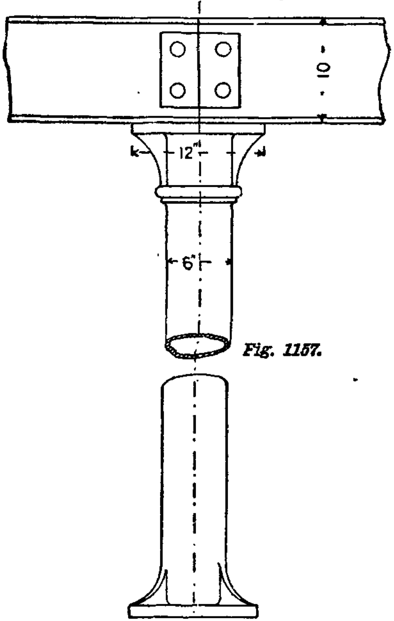


Fig. 1157.

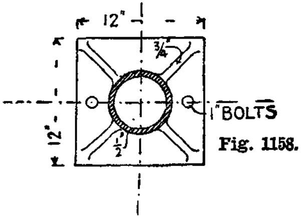
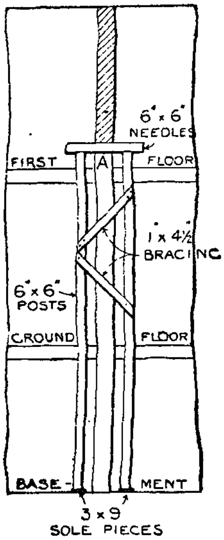


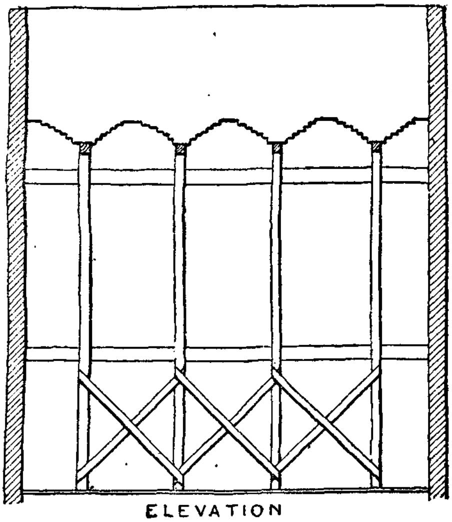
Fig. 1158.

Figs. 1157 and 1158.—Elevation and Sectional Plan of Column. Fig. 1159.—Plan of Girders at Junction. Fig. 1160.—Plan of Head of Column.

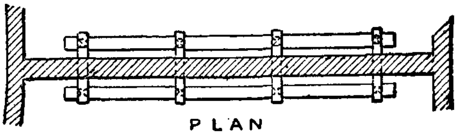
maximum of 15 tons to carry, with a total height of 10 ft. The ratio of 10 ft. length to 8 in. diameter will be $\frac{10 \times 12}{8} = 15$, and the safe load for that ratio equals 3 tons per square inch. Then $\frac{15 \text{ tons}}{3 \text{ tons}} = 5$ sq. in. sectional area, which will be obtained by a thickness of ¼ in. This is not enough for practical safety, and to get a good and an economical proportion either the thickness must be increased or the diameter (8 in.) must be reduced. Say 6 in. diameter (see Fig. 1157), ratio of length to diameter 20, safe load 2 tons per square inch, $\frac{15}{2} = 7\frac{1}{2}$ sq. in. area, which will be obtained by a thickness of $\frac{7}{16}$ in.; therefore make the columns 6 in. diameter and ½ in. thick, the head and base of columns to be as in Figs. 1157 and 1158. York stone template under columns 20 in. by 20 in. by 3 in. upon brick piers with footings below. Fig. 1159 is a plan of the girders at the junction; Fig. 1157 is elevation; Fig. 1158 plan at base; and Fig. 1160 plan at head.



SECTION
Fig. 1162.



ELEVATION
Fig. 1163.



PLAN
Fig. 1161.

Figs. 1161 to 1163.—Plan, Section, and Elevation of Posts and Needles.

Making Two Shops into One.

Assume the following case: Two adjoining ground-floor shops are to be made into one, without disturbing the business more than is inevitable. This entails removing a 14-in. party wall, 25 ft. long, and carrying a wall above on a wrought-iron girder supported by a cast-iron stanchion at each end, and a central cast-iron column standing on the party wall below the floor line of the shop. It is supposed that neither of the shop floors can be used for carrying temporary supports; the floors immediately above the shops are carried independently of the party wall. The following is the solution of the difficulty. A system of posts and needles is arranged as in plan (Fig. 1161). Holes are made in the shop floors where the posts are to go through, sufficiently wide to angle them in. The posts are then stood upon the sole pieces, holes are made in the party wall for the needles, and the posts wedged up to them and braced together in the basement, as shown in elevation by Figs. 1162 and 1163. Or the posts might be built up of two thicknesses of 3-in. by 9-in. deals bolted together. Then the brickwork between each pair of posts would be removed to enable some small bracing to be fixed in the other direction as in the section (Fig. 1162). This space would then be clear for the insertion of the stanchions and columns, and after these were in place the girder might be threaded through at A and fixed to them. Then the brickwork between the needles would be put in, the needles removed, and the holes made good.

17 Ft. + Section Cast-Iron Stanchion to Support Load of 50 Tons.

Fig. 1164 represents two views of a + section cast-iron stanchion to carry a safe load of 50 tons, length of column 17 ft. The approximate safe load on cast-iron stanchions 18 to 20 diameters long = (say) 2 tons per square inch. For 20 diameters long the width would be $\frac{17 \times 12}{20}$ = 10·2, and the required sectional area will be $\frac{50}{2}$ = 25 sq. in. These conditions will be provided by a stanchion of + section, about $10\frac{1}{2} \times 10\frac{1}{2} \times 1\frac{1}{4}$, with fillets in the angles; the section and elevation will be as shown in Fig. 1164. The strength of this stanchion by a table given in Fidler's "Bridge Construction" will be found as follows (see Fig. 1165):

l = length in inches.

r = radius of gyration in inches.

I = moment of inertia of cross-section.

A = area of cross-section in square inches.

$$\text{I} = \frac{\text{BD}^3 + 2\,bd^3}{12} = \frac{1\frac{1}{4} \times 10\frac{1}{2}^3 + 2 \times 4\frac{5}{8} \times 1\frac{1}{4}^3}{12}$$

$= 122{\cdot}09.$

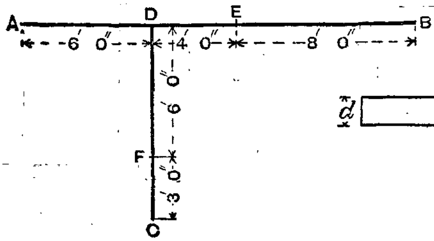


Fig. 1166.—Finding Distribution of Loads on Supports.

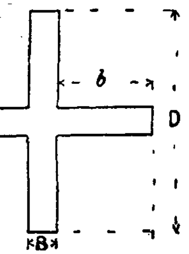


Fig. 1165.—Finding Strength of + Section Stanchion.

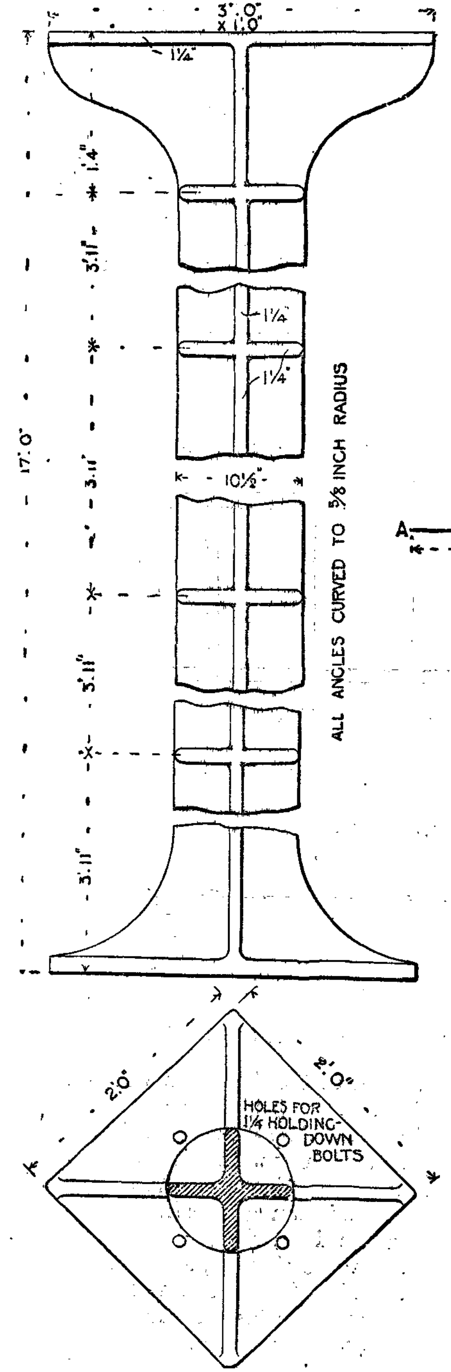


Fig. 1164.—Cast-Iron Stanchion to support Load of 50 Tons.

$\text{A} = \text{BD} + 2\,bd = 1\frac{1}{4} \times 10\frac{1}{2} + 2 \times 1\frac{1}{4} \times 4\frac{5}{8} = 24{\cdot}63.$

$$r^2 = \frac{\text{I}}{\text{A}} = \frac{122{\cdot}09}{24{\cdot}63} = 4{\cdot}96.$$

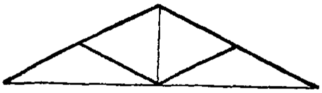


Fig. 1167.

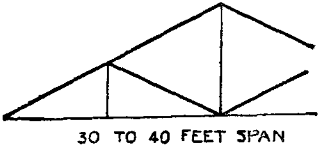


Fig. 1168.

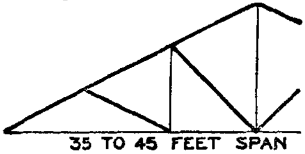


Fig. 1169.

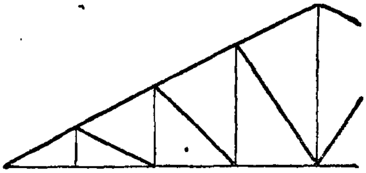


Fig. 1170.

Figs. 1167 to 1170.—Outlines of King-Rod Roof Trusses.

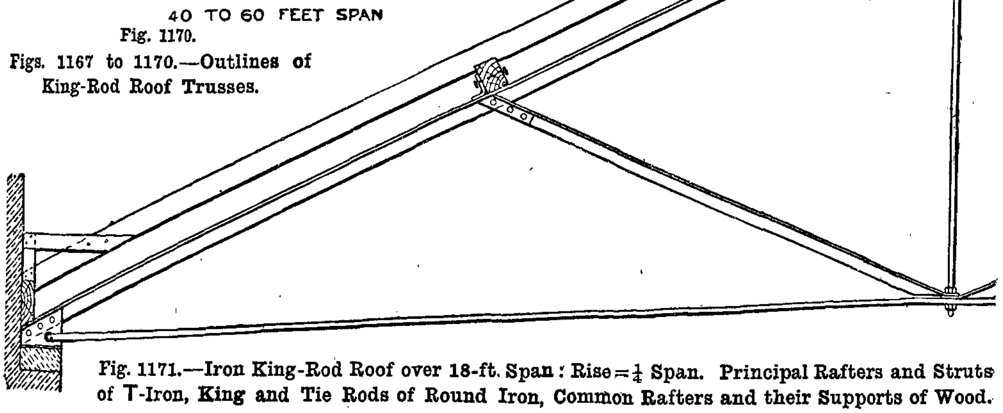


Fig. 1171.—Iron King-Rod Roof over 18-ft. Span: Rise = ¼ Span. Principal Rafters and Struts of T-Iron, King and Tie Rods of Round Iron, Common Rafters and their Supports of Wood.

$\therefore r = \sqrt{4{\cdot}96} = 2{\cdot}23.$

$\frac{l}{r}$ = ratio of length to radius of gyration $= \frac{17 \times 12}{2{\cdot}23} = 91{\cdot}4$. Fidler's table for cast-iron breaking weight gives, ends rounded, 17,600 lb. per square inch; ends fixed, 42,000 lb. per square inch, of which the mean is $\frac{29{,}800}{2240} = 12{\cdot}4$ tons per square inch. Factor of safety = 6, $\frac{12{\cdot}4}{6} = 2{\cdot}067$ tons per square inch, $24{\cdot}63 \times 2{\cdot}067 = 50{\cdot}9$ tons safe load.

Finding Distribution of Loads on Supports.

Assume a distributed load of 2 cwt. per foot run on A B, C D (Fig. 1166); load at F. 36 cwt.; load at E, 36 cwt. It is required to find the loads on A, B, and C.

This problem is solved in the following way:—

Load on C $= \frac{(6 + 3)^2}{2}$ = from distributed load ... ... 9 cwt.

Load on C $= \frac{36 \times 6}{6 + 3}$ = from concentrated load F ... ... 24 „

33 cwt.

Load on D $= \frac{(6 + 3)^2}{2}$ = from distributed load ... ... 9 cwt.

Load on D $= \frac{36 \times 3}{6 + 3}$ = from concentrated load F ... ... 12 „

Total D = 21 cwt.

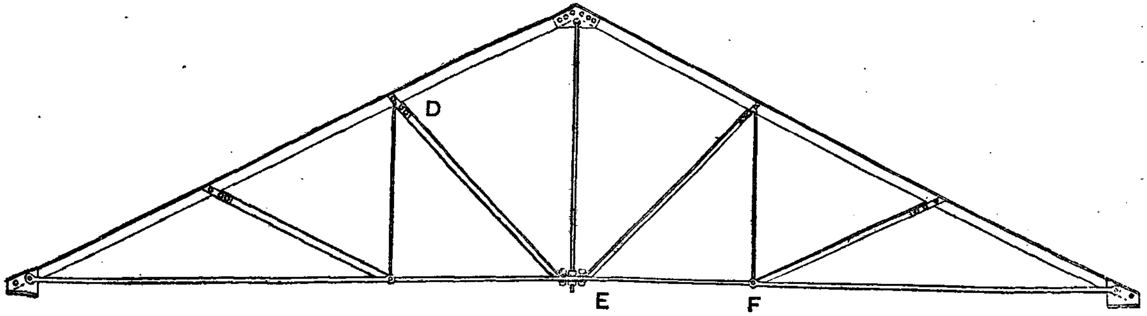


Fig. 1173.—King-Rod Roof, 35-ft. Span.

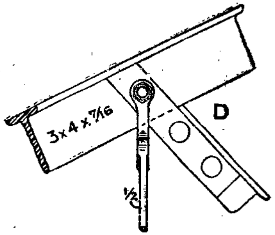


Fig. 1174.—Details of Joint at D (Fig. 1173).

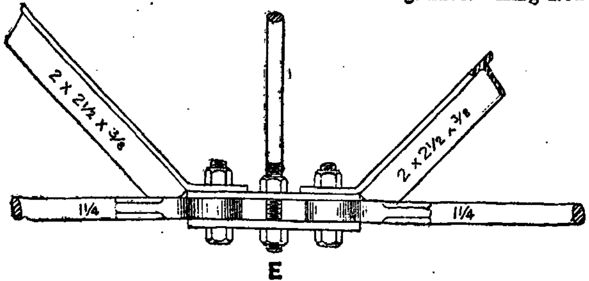
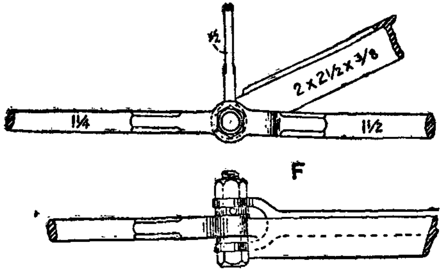
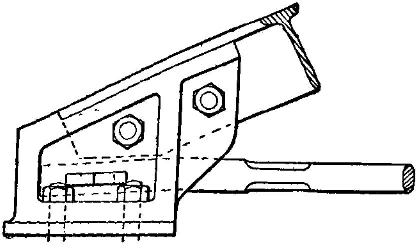

2 X 2½ X 3/8
2 × 2½ × 3/8
1¼
1¼
E

Fig. 1175.—Details of Joint at E (Fig. 1173).


½
2×2½×3/8
1¼
1½
F

Figs. 1176 and 1177.—Details of Joint at F (Fig. 1173).



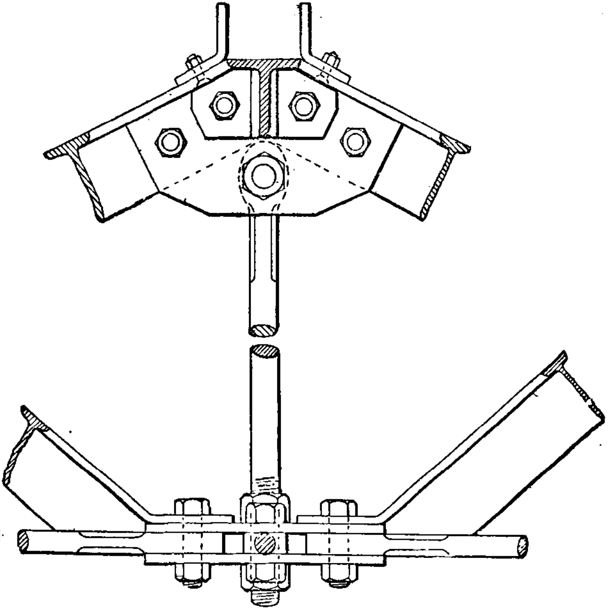


Fig. 1172.—Joints in King-Rod Truss.

Load on A $= \frac{(6 + 4 + 8)^2}{2} =$ from distributed load ... ... 18 cwt.

Load on A $= \frac{21\ (4 + 8)}{6 + 4 + 8} =$ from concentrated load D ... ... 14 „

Load on A $= \frac{36 \times 8}{6 + 4 + 8} =$ from concentrated load E ... ... 16 „

Total A = 48 cwt.

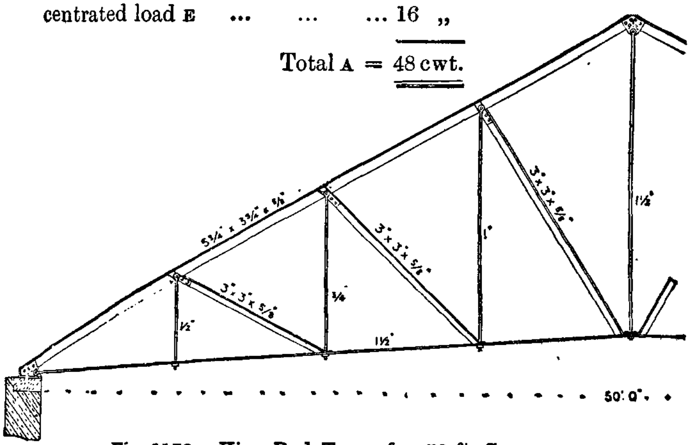


Fig. 1178.—King-Rod Truss for 50-ft. Span.

Load on B $= \frac{(6 + 4 + 8)^2}{2} =$ from distributed load ... ... 18 cwt.

Load on B $= \frac{21 \times 6}{6 + 4 + 8} =$ from concentrated load D ... ... 7 „

Load on B $= \frac{36\ (6 + 4)}{6 + 4 + 8} =$ from concentrated load E ... ... 20 „

Total = 45 cwt.

Answer :—

Load on A = 48 cwt.
„ B = 45 „
„ C = 33 „
126 cwt.

Proof :—

$$\begin{array}{ll} \text{AB} \left\{ \begin{array}{l} 6 \times 2 = 12 \text{ cwt.} \\ 4 \times 2 = 8 \\ 8 \times 2 = 16 \end{array} \right. \\ \text{CD} \left\{ \begin{array}{l} 6 \times 2 = 12 \\ 3 \times 2 = 6 \end{array} \right. \\ \text{F} \quad = 36 \\ \text{E} \quad = 36 \end{array}$$

Total 126 cwt.

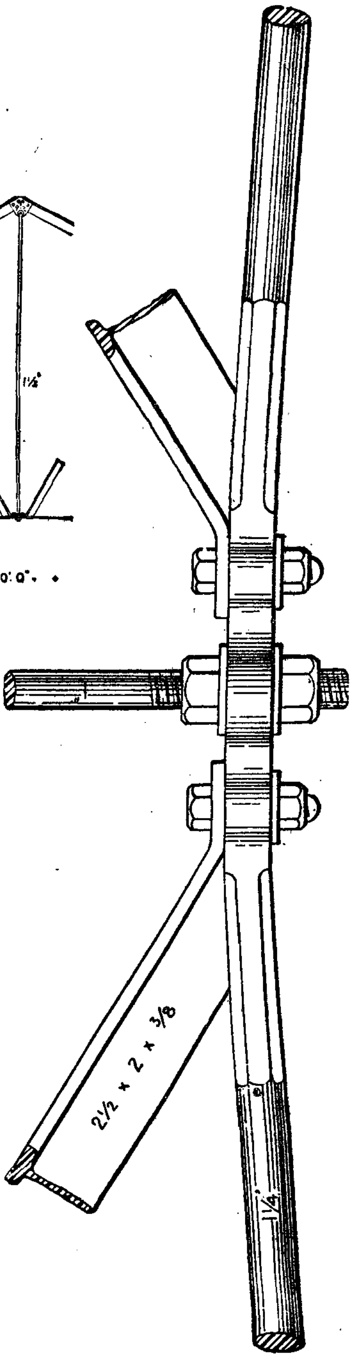


Fig. 1179.—Joint at Foot of King-Rod.

Protecting Iron Columns from Fire.

Iron columns may be protected from fire by surrounding them with expanded metal lathing and covering them with plaster, pressing it through the space to form a good key. Another method is to surround the column with moulded blocks of terra-cotta keyed for plastering, or with pumice concrete blocks. Any artificial fire-resisting material which can be incorporated with selenitic plaster may be used in the same way. Instructive diagrams in this connection are given on pp. 170 and 171.

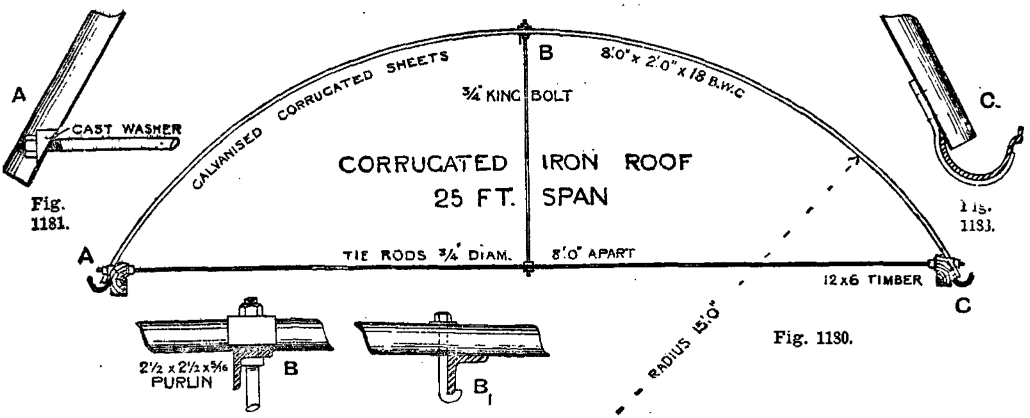

A
CAST WASHER
GALVANISED CORRUGATED SHEETS
B
8'0" x 2'0" x 18 B.W.G
3/4 KING BOLT
CORRUGATED IRON ROOF
25 FT. SPAN
C.
A
TIE RODS 3/4 DIAM. 8'0" APART
12x6 TIMBER
C
RADIUS 15'0"
2½ x 2½ x 5/16 PURLIN
B
B1

Fig. 1181.

Fig. 1183.

Fig. 1180.

Fig. 1182.

Fig. 1184.

Figs. 1180 to 1184.—King-Bolt Corrugated Iron Roof with Details of Joints.

Designing Wrought-Iron Roof Trusses.

There is no single formula available for designing wrought-iron roof trusses. The type of truss (varying chiefly with the span) has first to be decided on, and so arranged that the principal rafters are supported at intervals of about 8 ft. Then a frame diagram has to be drawn out, and a reciprocal stress diagram constructed, from which the stresses in each member may be measured off. The form of cross section of each is then decided, approximate dimensions being fixed and the strength calculated, altering the dimensions as may be necessary until the strength agrees with the stress that is to be resisted. Then each joint has to be designed so that the connections are at least as strong as the parts that have to be connected. The complete design of a truss may then be made, and, in the case of a hipped roof, the arrangement of the hip and jack trusses settled. The whole subject of stresses in roofs and other structural work is fully discussed later in this book.

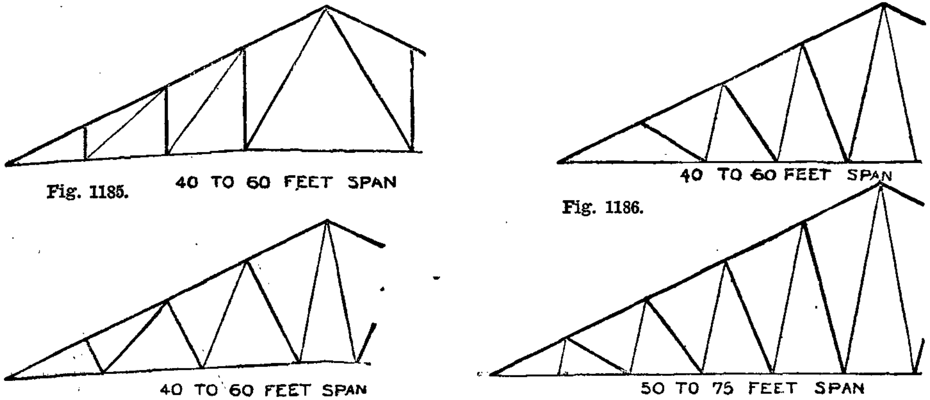

40 TO 60 FEET SPAN
40 TO 60 FEET SPAN
40 TO 60 FEET SPAN
50 TO 75 FEET SPAN

Fig. 1185.

Fig. 1186.

Fig. 1187.

Fig. 1188.

Figs. 1185 to 1188.—Outlines of Queen-Rod Roof Trusses.

Figs. 1189 to 1192.—Queen-Rod Roof Truss for 50-ft. Span, with Details of Joints.

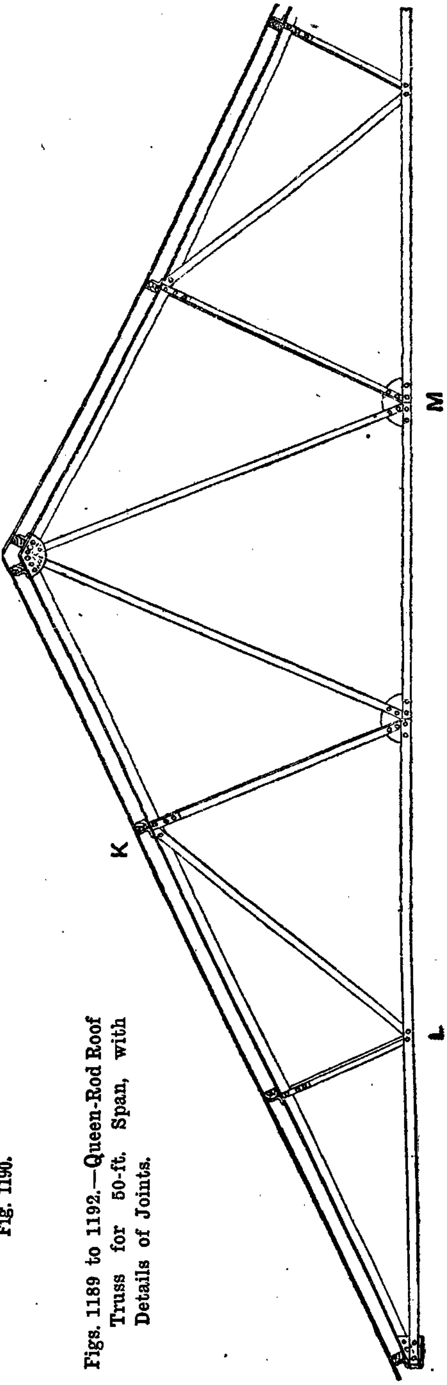


Fig. 1189.

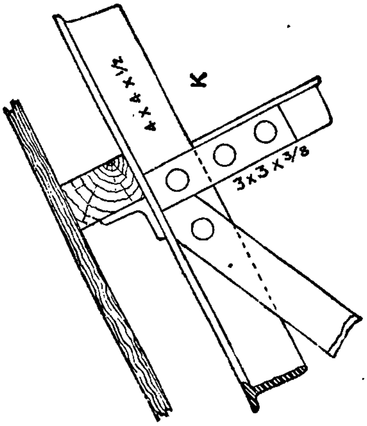

K
4 x 4 x 1/2
3 x 3 x 3/8

Fig. 1190.

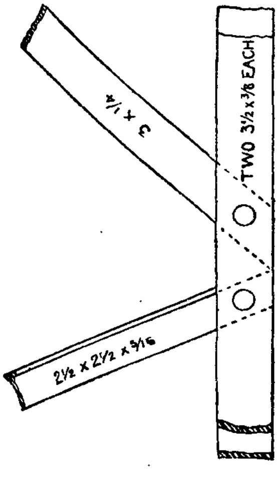

L
3 x 1/4
TWO 3 1/2 x 3/8 EACH
2 1/2 x 2 1/2 x 5/16

Fig. 1191.

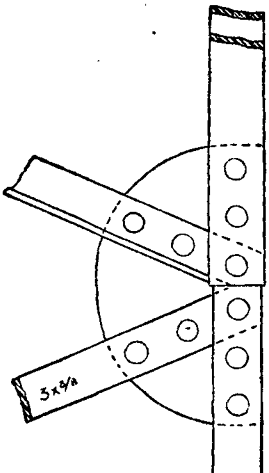


Fig. 1192.

Determining Sizes of Members in Iron Roof Truss.

A common approximate rule is to allow 5 tons per square inch of net section in tension and 3 tons per square inch in compression; as a guard against defective welds some allow only 4 tons in tension. In compression the stress should vary with the ratio of length to diameter, as by Gordon's formula, or the ratio of length to radius of gyration, as in Rankine's modification, the co-efficients varying with each formula and with the form of section.

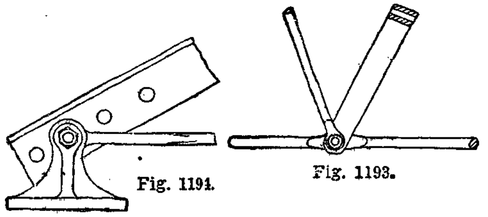


Fig. 1194.
Fig. 1193.

Figs. 1193 and 1194.—Forms of Joints when Tie-Bar and Tension Members are of Round Rod.

Lean-to Iron Roof.

A lean-to iron roof (corrugated iron) can be used for an excessively wide span if the trusses are designed in accordance with the requirements. The largest span that can be covered without trussing does not exceed 30 ft.; 10-in. by 5-in. rolled joist principals, 8 ft. apart, and 3 in. by 3-in. by ⅜-in. angle-iron purlins may be used up to that spacing. With 3-in. by 3-in. by ⅜-in. angle-iron principals, 3 ft. apart, and no purlins, the span must not exceed say 8 ft.

King-Rod Roof Trusses.

Four outlines of typical king-rod trusses are presented by Figs. 1167 to 1170. The simplest of these is shown in detail by Fig. 1171 (the joints in which could be as in Fig. 1172), and one suitable for a span of from 35 ft. to 45 ft. by Fig. 1173, details of the joints at D, E, F being shown by Figs. 1174 to 1177. The truss illustrated by Fig. 1178 is suitable for a 50-ft. span, a truss of this outline being available for any span between 40 ft. and 60 ft. Another elevation of the joint at the foot of a king-rod is given by Fig. 1179; the king-rod passes through the tie-rod and has a screw-nut adjustment. In all the outline diagrams of iron trusses in this section the thick lines represent compression and the thin lines tension.

King-Bolt Corrugated Iron Roof.

A semicircular roof composed of galvanised corrugated sheets supported by king-bolt and tie-rods is illustrated by Fig. 1180. Full details of the joints indicated at A, B, and C are shown in Figs. 1181 to 1183, an alternative method of making joint B being shown by Fig. 1184.

Strength of Corrugated Sheets.

The formula for strength of corrugated-iron sheets is $W = \frac{892\,tbd}{l}$, where l = unsupported length of plate in inches, t = thickness of plate in inches, b = breadth of plate in inches, d = depth of corrugations in inches, W = breaking weight distributed in hundredweights. In the case of a sheet 2 ft. wide, with eight corrugations 1½ in. deep, and 5 ft. long between the bearings, the ends fixed, the load being equally distributed over the 10 ft. super., $l = 60$, t = say, 18 S.S.G. = ·0495 in., $b = 24$, $d = 1·5$, then $W = \frac{892 \times ·0495 \times 24 \times 1·5}{60} = 26\frac{1}{2}$

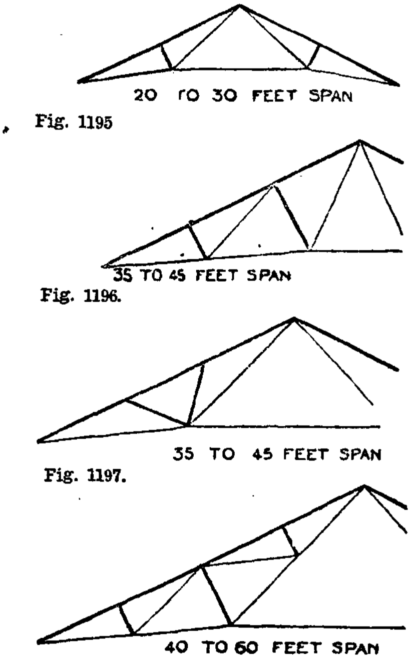


Fig. 1195
Fig. 1196.
Fig. 1197.
Fig. 1198.

Figs. 1195 to 1198.—Outlines of Trussed Rafter Iron Roofs.

cwt., or 2·65 cwt. per ft. super., which, with a factor of safety of 5, would give about ½ cwt. per ft. super. safe load. Galvanising is said not to reduce the tensile strength of wrought iron. Galvanised sheets always fail by corrosion caused by the acids and moisture in the atmosphere, therefore the thinner the sheet the shorter its life and the higher the factor of safety necessary. The fixing of the ends is so slight that no addition to the strength can be assumed on that account. The above is a special formula quoted by Molesworth, but the calculation for thicker sheets, such as are used in bridge floors, might be made by finding the moment of inertia.

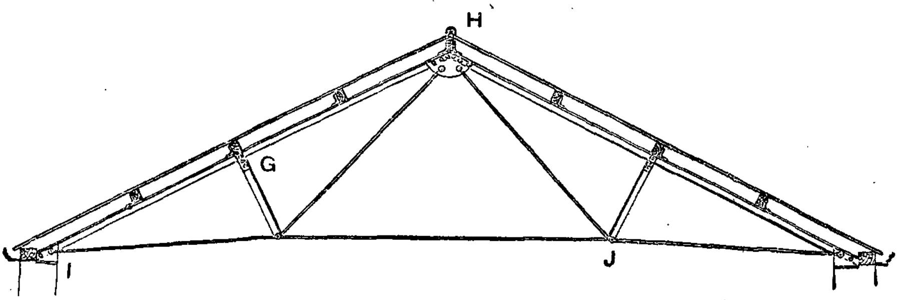


Fig. 1199.

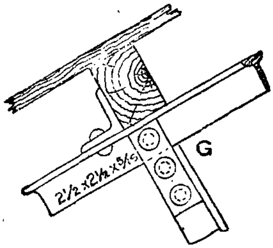


Fig. 1200.

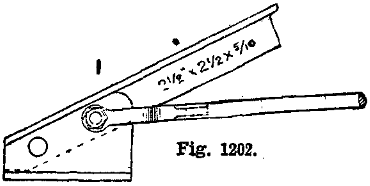


Fig. 1202.

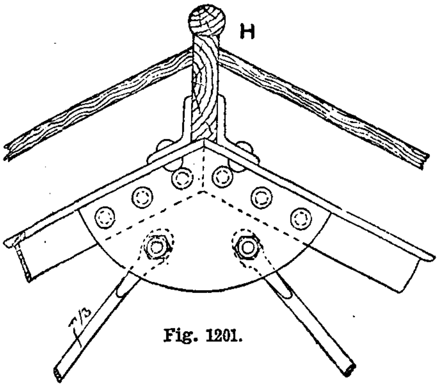


Fig. 1201.

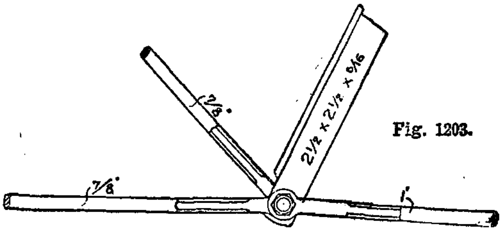


Fig. 1203.

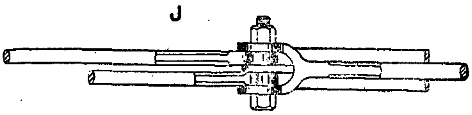


Fig. 1204.

Figs. 1199 to 1204.—Trussed Rafter Iron Roof for 25-ft. Span, with Details of Joints.

Queen-Rod Roof Trusses.

Diagrams presenting outlines of typical queen-rod roof trusses are given by Figs. 1185 to 1188. An elevation of a queen-rod truss designed for a span of 50 ft. is shown by Fig. 1189, details of the joints at K, L, M being given by Figs. 1190 to 1192. The joint at M (Fig. 1189) may be as in Fig. 1193 when the member in tension and the tie-bar are round rods; in this case, the joint of the foot of the principal rafter with the end of the tie-bar will be as in Fig. 1194.

Trussed Rafter Iron Roofs.

Outlines of typical trussed rafter iron roofs are shown by Figs. 1195 to 1198. An elevation

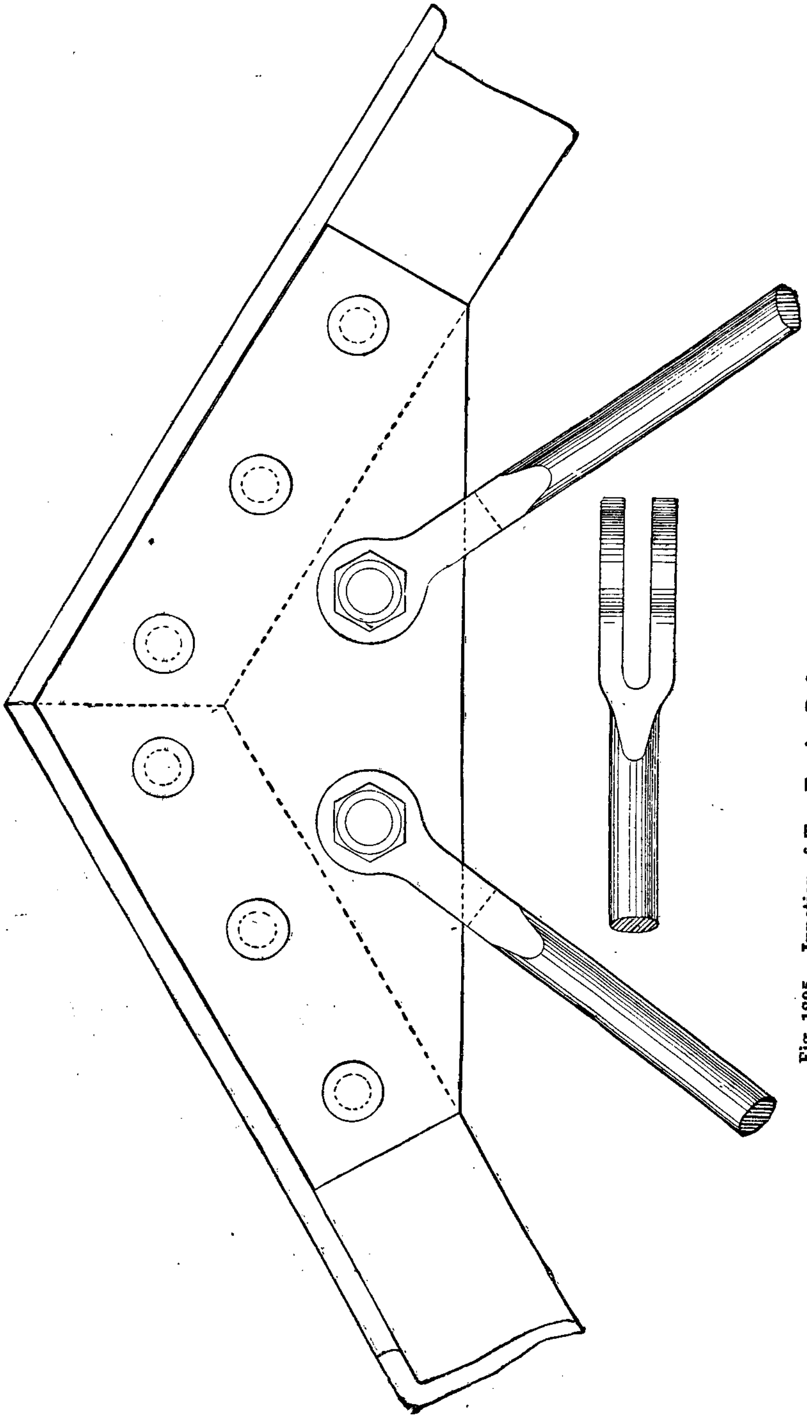


Fig. 1205.—Junction of Two Tension Rods with Principal Rafters at Apex.

of a truss composed of iron bars and rods is given by Fig. 1199, whilst Figs. 1200 to 1204 show details of the joints at G, H, I, J. The important joint of the two tension rods with the principal rafter at its apex is shown enlarged by Fig. 1205, the forked end of one of the rods being shown separately. A roof truss of the same outline as Fig. 1199, but composed of T-iron rafters and bar-iron tie-rod, struts, and ties, is illustrated by Fig. 1206; details of the necessary details are given by Figs. 1211 to 1214, 1211 and 1212 showing the joint of rafter and tie-bar; Fig. 1213 joint of tie-bar, struts and sloping tie; and Fig. 1214 joint of ties and principal rafter at the top.

More Elaborate Trussed Rafter Roofs.

The half of a trussed rafter roof known as the Fink is illustrated by Fig. 1215, sections of the various members also being given. The

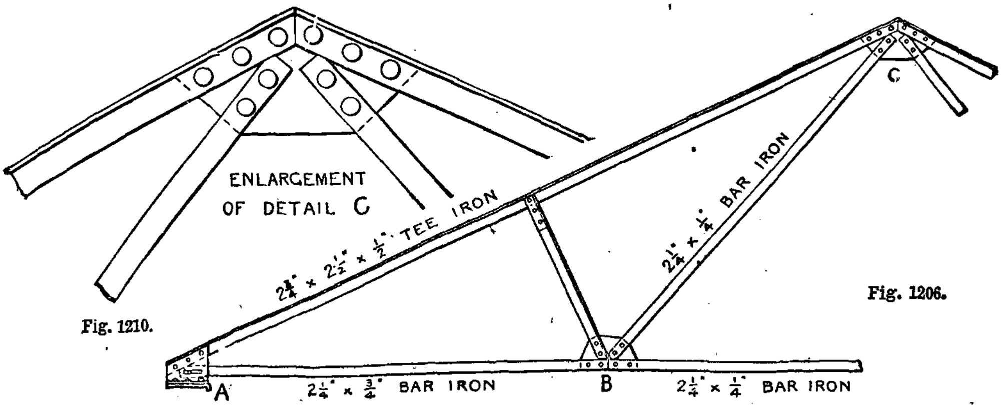


Fig. 1210.

Fig. 1206.

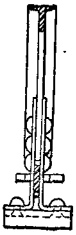


Fig. 1207.

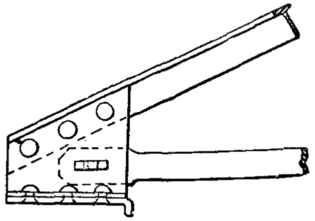


Fig. 1208.

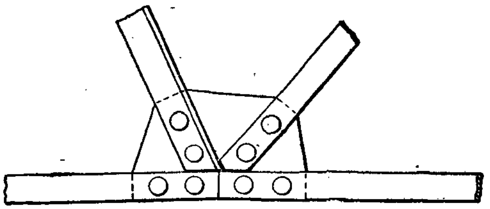


Fig. 1209.

Figs. 1206 to 1210.—Wrought-Iron Trussed Rafter Roof for 30-ft. Span, with Details of Joints.

the joints at A, B, C are given on a larger scale by Figs. 1207 to 1210. The truss shown by Fig. 1206 is of wrought iron and designed for a 30-ft. span. The following dimensions are suitable, and are taken from Molesworth's "Pocket Book of Engineering Formulæ": Principal rafter, 2¾-in. by 2½-in. by ½-in. T-iron; outer portion of tie-rod, 2¼-in. by ⅜-in. bar; middle tie, 2¼-in. by ¼-in. bar; inner tie 2¼-in. by ¼-in. bar. A truss agreeing with the outline given by Fig. 1197 could be constructed for a span of 35 ft. with T-iron rafters, angle-iron struts, and flat bar ties or tension members; joints at A, B, C are shown in detail by Figs. 1216 to 1219. A similar truss is shown by Fig. 1220, this being dimensioned to show the scantlings of the members for a 40-ft. span. The truss shown in outline by Fig. 1221 is a good form for an open roof of wrought iron or mild steel, but it would be unsuitable for the attachment of a ceiling owing to its camber. However, a Building Construction examination question has required the weight per foot of the various members of such a truss to be given. Assuming that the hanging loads indicated in Fig. 1221 represent the weight of an attached ceiling, it is one of the lightest forms of truss for the span owing to the shortness of the compression members, but it is more expensive than some other forms, owing to the number of joints. The weights in pounds per foot run are found by multiplying the sectional areas by 3⅓, and consequently considerable labour would have to be expended in calculation to determine the proper sectional area of each member to suit the length, stress, and factor of safety after

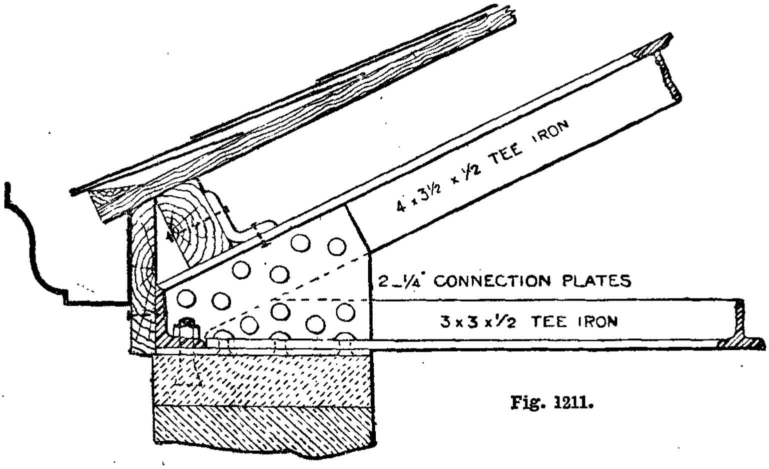

4 x 3½ x ½ TEE IRON
2-¼" CONNECTION PLATES
3 x 3 x ½ TEE IRON

Fig. 1211.

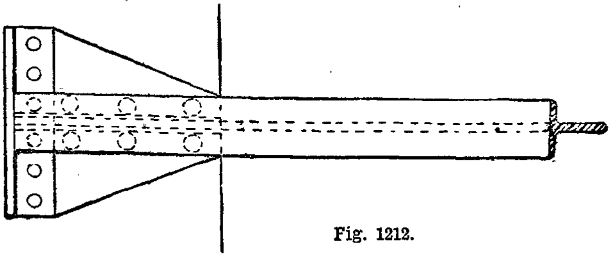


Fig. 1212.

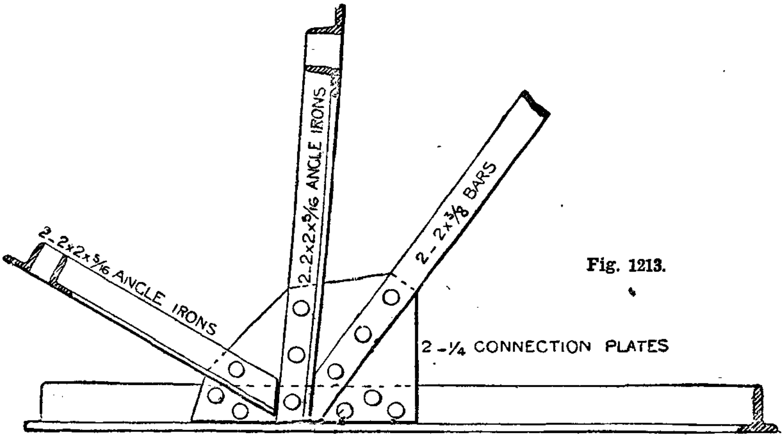

2-2x2x5/16 ANGLE IRONS
2-2x2x5/16 ANGLE IRONS
2-2x3/8 BARS
2-¼ CONNECTION PLATES

Fig. 1213.

Figs. 1211 to 1213.—Enlarged Details of Joints of Trussed Rafter Iron Roof, shown in Outline by Fig. 1197 (p. 314).

Fig. 1214.—Joint at Apex of Trussed Rafter Iron Roof, shown by Fig. 1197 (p. 314).

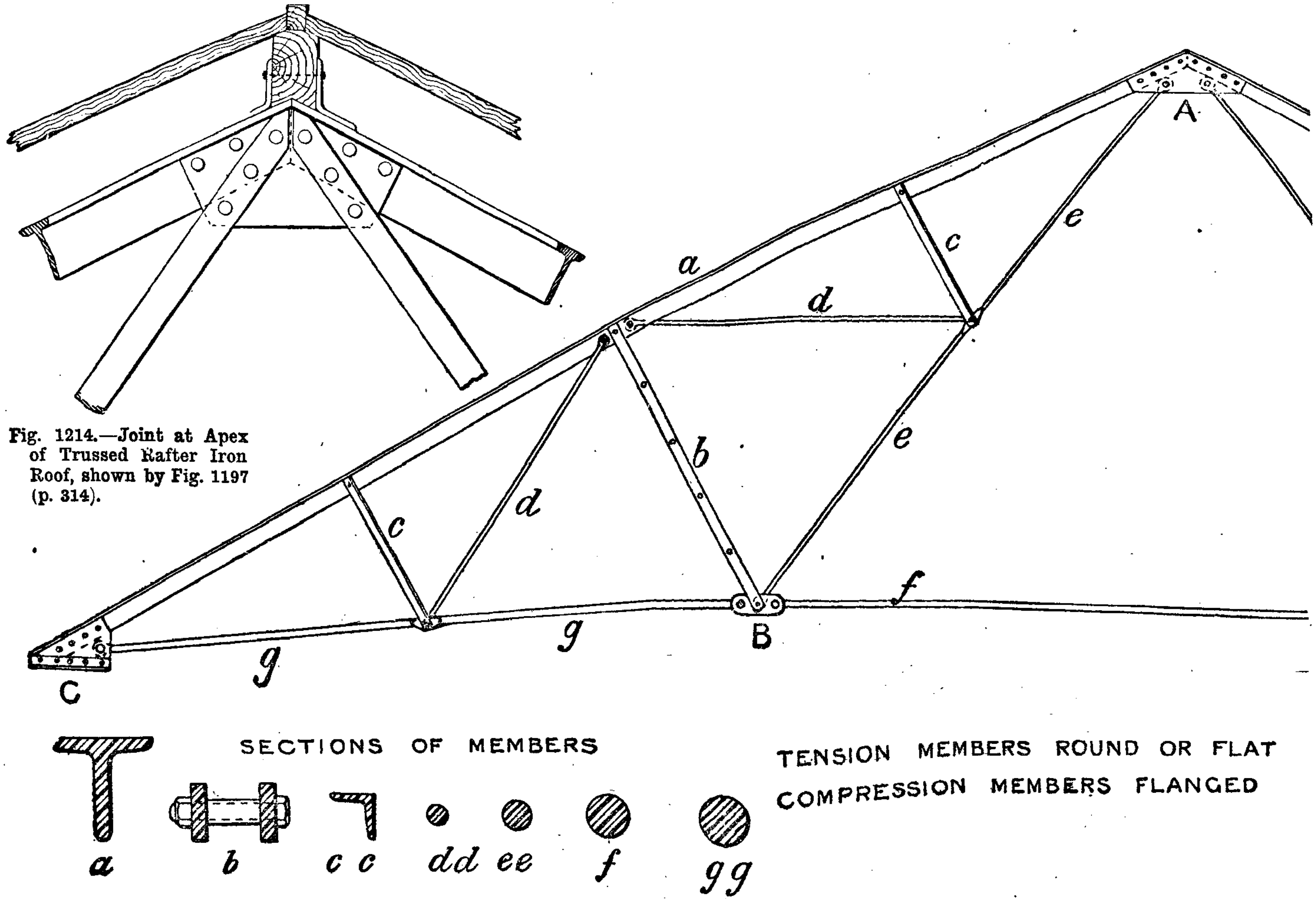
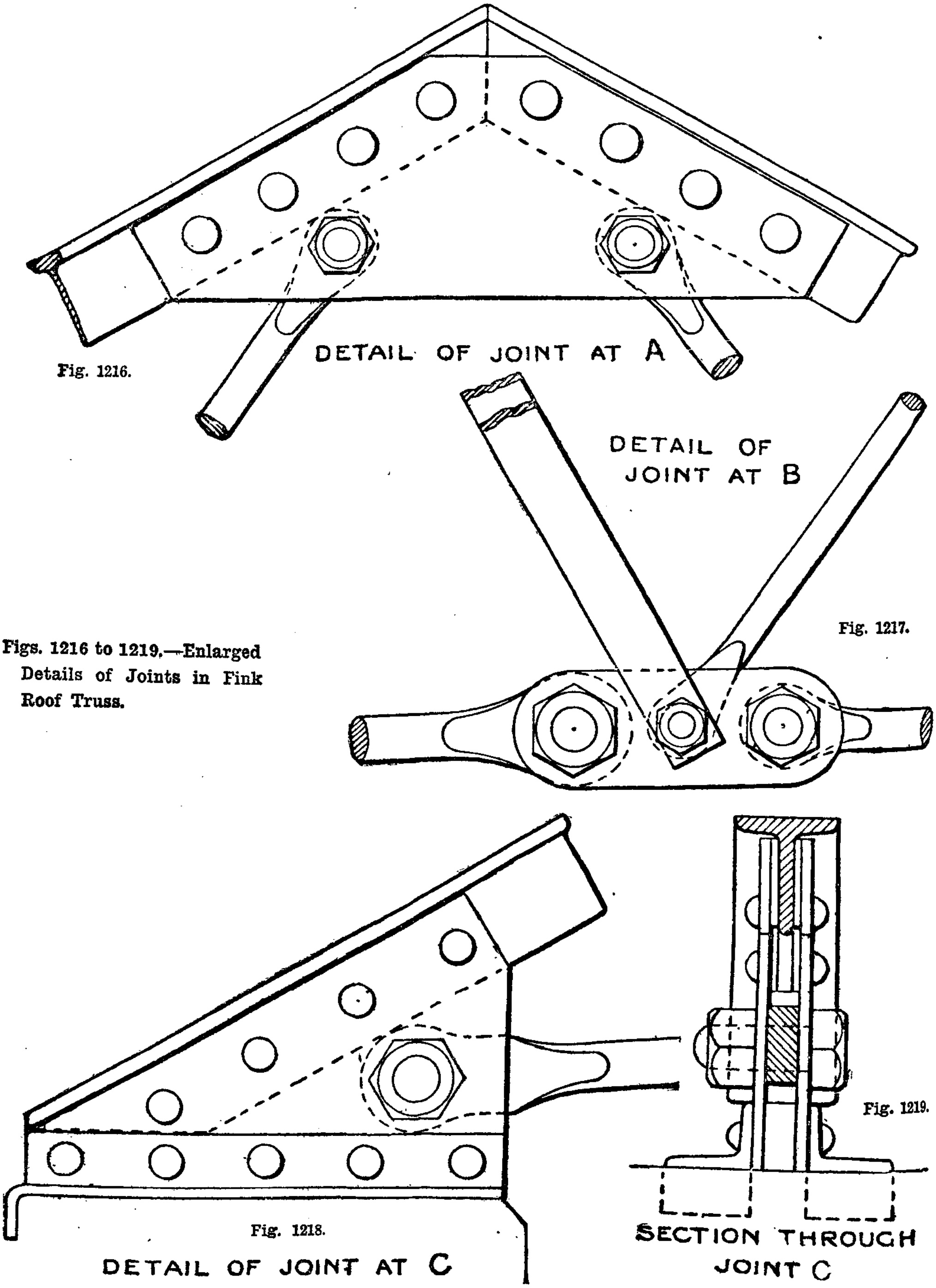

A
a
c
e
d
b
e
d
c
f
C
g
g
B
SECTIONS OF MEMBERS
TENSION MEMBERS ROUND OR FLAT
COMPRESSION MEMBERS FLANGED
a
b
c c
dd
ee
f
gg

Fig. 1215.—Fink Trussed Rafter Roof for 48-ft. Span.



DETAIL OF JOINT AT A

Fig. 1216.

DETAIL OF JOINT AT B

Fig. 1217.

Fig. 1218.

DETAIL OF JOINT AT C

Fig. 1219.

SECTION THROUGH JOINT C

Figs. 1216 to 1219.—Enlarged Details of Joints in Fink Roof Truss.

deducting rivet-holes. The figures following are approximate only. Joint A (see also Fig. 1222): For the 22¼ tons at 4 tons per square inch compression = say 5½ sq. in. area, or a 6-in. by 4-in. by ⅝-in. or 19-lb. T, or two 4-in. by 4-in. by $\frac{7}{16}$-in. or 11-lb. angles. For the 21 tons at 5 tons per square inch gross area tension = say 4¼ sq. in. area, or two 4-in. by $\frac{9}{16}$-in. or 7½-lb. bars. ¾-in. steel rivet = 3 tons single shear, $\frac{22\frac{1}{4}}{3}$ = 7·44, say 8 rivets required. Joint B (see also Fig. 1223): For 5⅝ tons at 4 tons per square inch compression = say 1½ sq. in. required, but would not be made less than 3-in. by 3-in. by ⅜-in. or 7-lb. angle. For the 9 tons at 5 tons gross for tension = 1·8 sq. in. = 4-in. by ½-in. or 7-lb. bar. For the 8 tons tension say a similar bar. For the 16 tons tension say two similar bars, the connections being made as shown in the diagram. The stresses in the various members were found by a method to be shown later on.

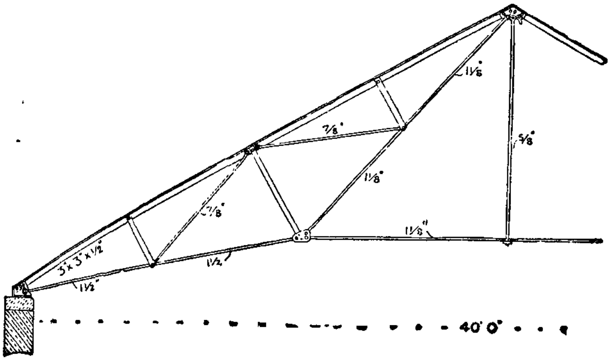

1⅛
⅝
⅞
1⅛
⅞
1⅛
5x3x½
1½
1½
40'0"

Fig. 1220.—Fink Roof Truss for 40-ft. Span.

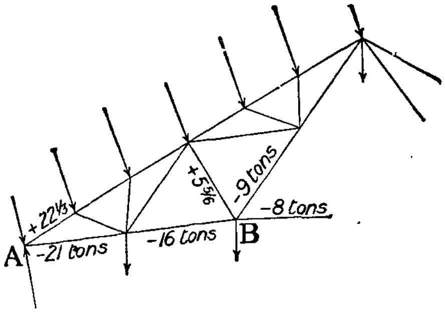

+22⅓
+5⅝
-9 tons
-8 tons
A
-21 tons
-16 tons
B

Fig. 1221.—Outline of Wrought-Iron Truss for Open Roof.

Steel Truss with Curved Soffit.

Fig. 1224 shows a general outline of the truss with a curved soffit over a span of 32 ft. The stresses for this roof will be found worked out towards the end of this book. Details of the joints indicated at A to F are shown by Fig. 1225.

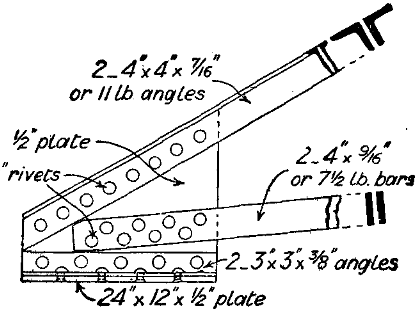

2_4"x4"x 7/16"
or 11 lb angles
½" plate
¾" rivets
2_4"x 9/16"
or 7½ lb. bars
2_3"x3"x ⅜" angles
24"x 12"x ½" plate

Fig. 1222.—Detail of Joint A (Fig. 1221).

Braced Iron Roof.

The design illustrated by Fig. 1226 is for a braced roof suitable for a span of 80 ft. It is similar to one made by Walter Jones, of Bow Bridge, London.

Iron Truss Supported on Cast-Iron Columns.

When an iron roof rests on cast-iron columns, the connection may be as in Fig. 1227. Here the principal rafter is a T iron 3½ in. by 5 in. by ⅝ in., and the tie-rod, 1½ in. in diameter, is capable of being tightened up.

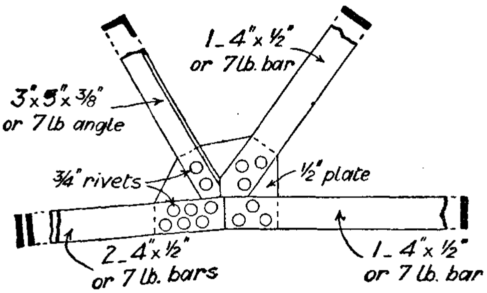

1_4"x ½"
or 7 lb. bar
3"x5"x ⅜"
or 7 lb angle
¾" rivets
½" plate
2_4"x ½"
or 7 lb. bars
1_4"x ½"
or 7 lb. bar

Fig. 1223.—Detail of Joint B (Fig. 1221).

Girders and Columns for Carrying Roof.

A slate roof (*between two warehouses*), *50-ft.* span and 30 ft. high, with a clear headway of

16 ft., cannot be carried by rolled steel joists, as the span is too great. There should be four cast-iron E stanchions about 8 in. by 6 in. by $\frac{3}{4}$ in., with proper cap and base and good foundations, two steel lattice girders in fifteen bays, with a depth of 3 ft. 6 in., and each capable of carrying with safety 20 tons distributed, and six king- or queen-post trusses if of wood, or of trussed rafter design if of iron, with the usual purlins, etc. The arrangement is shown in Fig. 1228.

Cambering Tie-Rod of Iron Roof.

The advantages of cambering the tie-rod of an iron roof are—better appearance, increased headway, shorter struts, and increased stresses on light roofs, enabling the material to be more nearly proportioned to the stress. In small iron roof trusses there are often some members of greater sectional area than the stress requires, in order that the pieces may not look absurdly small and be liable to rust away. It should be noted that a $\frac{1}{20}$-in. depth of rust, which would

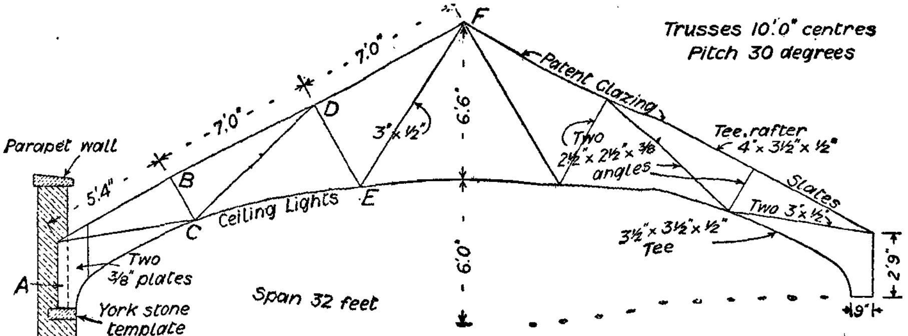

F
Trusses 10'0" centres
Pitch 30 degrees
7'0"
7'0"
5'4"
D
B
Parapet wall
3"x½"
6'6"
Patent Glazing
Two 2½"x 2½"x ⅜" angles
Tee.rafter 4"x 3½"x ½"
Slates
Two 3"x ½"
C
Ceiling Lights
E
3½"x 3½"x ½" Tee
Two ⅜" plates
A
6'0"
2'9"
Span 32 feet
York stone template
9"

Fig. 1224.—Outline of Steel Truss with Curved Soffit.

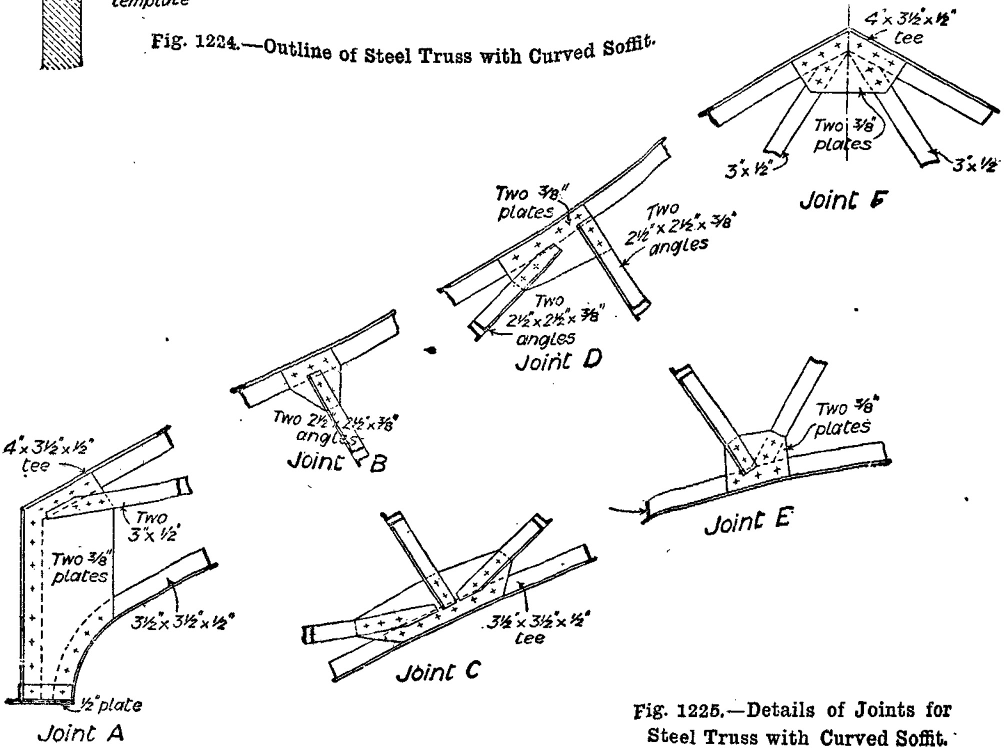

4"x 3½"x ½" tee
Two ⅜" plates
3"x ½"
3"x ½"
Joint F
Two ⅜" plates
Two 2½"x 2½"x ⅜" angles
Two 2½"x 2½"x ⅜" angles
Joint D
Two 2½"x 2½"x ⅜" angles
Joint B
Two ⅜" plates
Joint E
4"x 3½"x ½" tee
Two 3"x ½"
Two ⅜" plates
3½"x 3½"x ½"
½" plate
Joint A
3½"x 3½"x ½" tee
Joint C

Fig. 1225.—Details of Joints for Steel Truss with Curved Soffit.

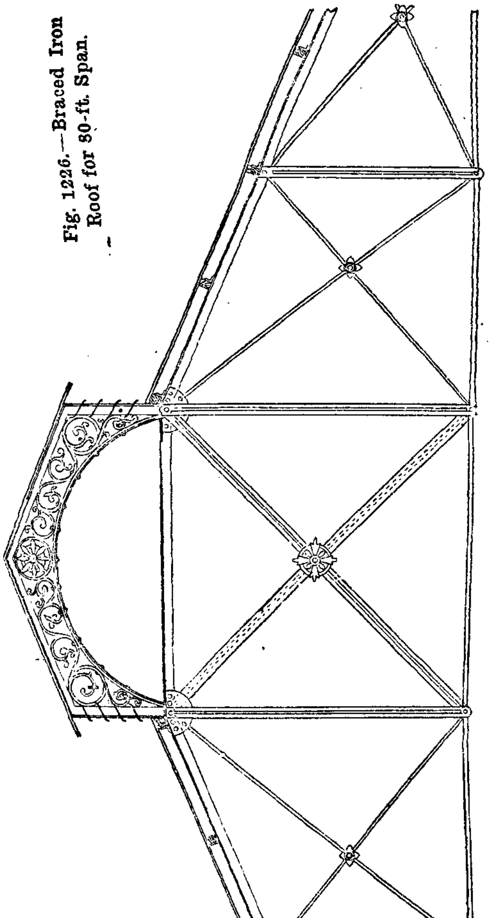


Fig. 1226.—Braced Iron Roof for 80-ft. Span.

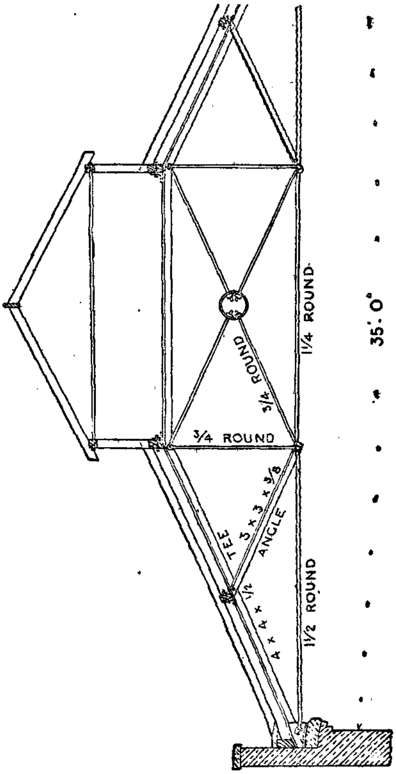

3/4 ROUND
1/4 ROUND
3/4 ROUND
TEE
3 x 3 x 3/8 ANGLE
4 x 4 x 1/2
1/2 ROUND
35' 0"

Fig. 1229.—Lantern Light in Steel Roof Truss.

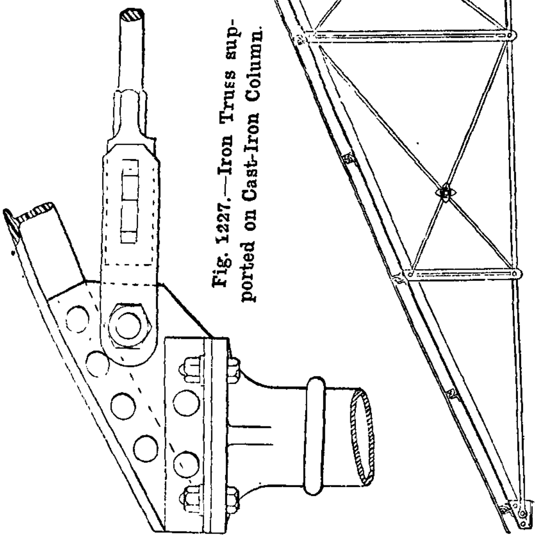


Fig. 1227.—Iron Truss supported on Cast-Iron Column.

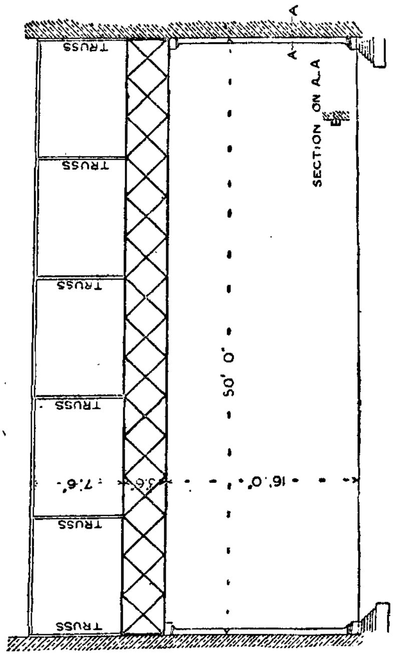

TRUSS
TRUSS
TRUSS
TRUSS
TRUSS
TRUSS
A
A
SECTION ON A.A
50' 0"
7'.6"
3'.6"
16'.0"

Fig. 1228.—Roof carried by Girders and Columns.

take away 15 per cent. of the strength of a 4-in. by $\frac{3}{4}$-in. bar, would reduce the strength of a 2-in. by $\frac{3}{8}$-in. bar by 30 per cent. in the same time.

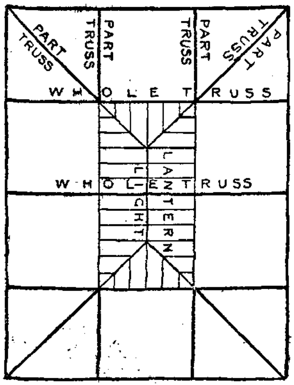


Fig. 1230.—Hipped Ends of Lantern Light in Steel Roof Truss.

Thickness of Joint Plates in Iron Roof.

Joint plates are generally proportioned according to the draughtsman's personal judgment. They should not be less than $\frac{1}{4}$ in. thick, and are not usually less than $\frac{3}{8}$ in., and not often thicker than $\frac{1}{2}$ in., even in large roof trusses. The combined thickness of the plates should be such that the bearing surface for the rivets is at least equal to the bearing surface in the bar between them, and the net sectional area through the shortest line of fracture should be at least equal to the net sectional area of bar, but these proportions are usually exceeded.

Allowance for Expansion of Metal in Fixing Iron Roof.

Allowance for expansion or contraction is not usually made in a roof of 50-ft. span; the extreme range of movement between summer and winter temperatures is about $\frac{7}{16}$ in. in 100 ft. In the case of a straight rigid structure like a girder, the ends of which had been securely fixed during one extreme of the temperature, the girder would, during the opposite extreme of temperature, undergo a stress of about 5 tons per square inch in the endeavour to overcome the resistance; but a roof truss can, on a rise of temperature, rise in the centre, and so relieve itself.

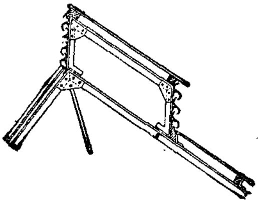


Fig. 1232.—Ventilator in Northern Light Roof.

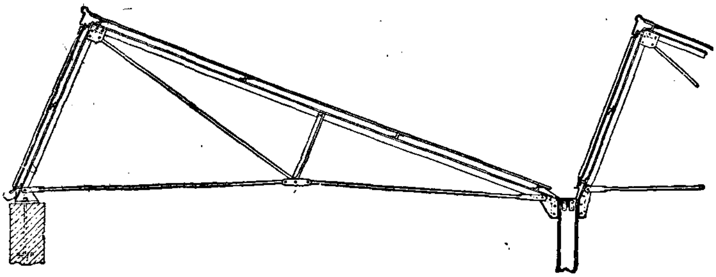
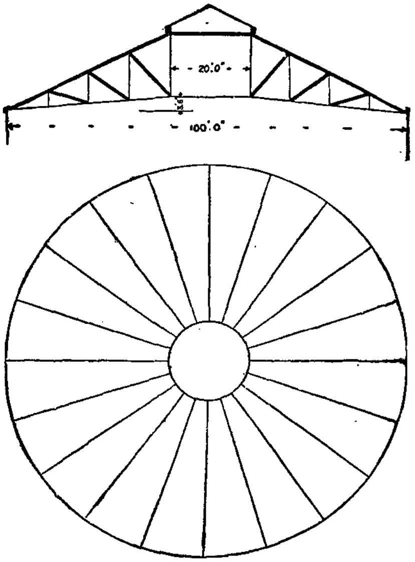


Fig. 1231.—Northern-Light, Saw-Tooth, or Weaving-Shed Roof.

Lantern Light in Steel Roof Truss.

A design for a steel roof truss of 35-ft. span, with lantern light, is presented by Fig. 1229, this figure showing the elevation of one truss in the cross-section through roof, whilst Fig. 1230 shows the arrangement of the hipped ends. The roof is arranged in three bays of 11 ft. 8 in., and so that will be the best distance



Figs. 1233 and 1234.—Roof for Circular Engine Shed.

apart for the trusses; and at each hip there will be two part trusses formed like one side of the main truss to meet the main truss at the end of the lantern.

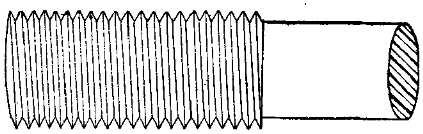


Fig. 1235.—Plus Thread on Tie-Rod.

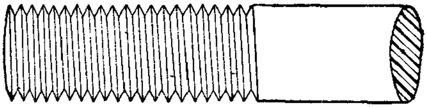
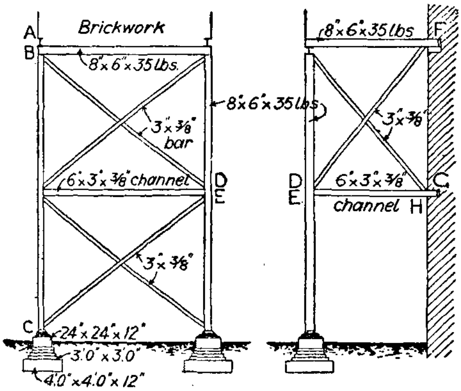


Fig. 1236.—Minus Thread on Tie-Rod.

Northern-Light, Saw-Tooth, or Weaving-Shed Roof.

A northern-light, or saw-tooth, or weaving-shed roof for 20-ft. span is shown in Fig. 1231, as made by A. and J. Main and Co., Limited, Possilpark, Glasgow. The main slope is covered with galvanised corrugated iron on angle-steel purlins, carried by rolled steel valley columns at 12 ft. 6 in. apart. The same truss may be used for a covering of slates or tiles with a closer spacing of the columns. A ventilator, as Fig. 1232, may be fixed to the roof if required.


A
B
Brickwork
8"x6"x35 lbs.
8"x6"x35 lbs.
3"x3/8" bar
6"x3"x3/8" channel
D
E
3"x3/8"
C
24"x24"x12"
3.0"x3.0"
4.0"x4.0"x12"
8"x6"x35 lbs
F
3"x3/8"
D
E
6"x3"x3/8" channel
G
H

Figs. 1237 and 1238.—Front and Side Elevations of Structure to carry Organ Chamber.

Roof for Circular Engine Shed.

A circular engine shed (100 ft. diameter) is to have a roof which will have no support except from the 18-in. walls, and is to be lighted from the top only. To roof a circular

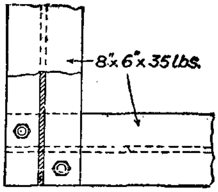


Fig. 1239.—Plan of Joists at Junction.

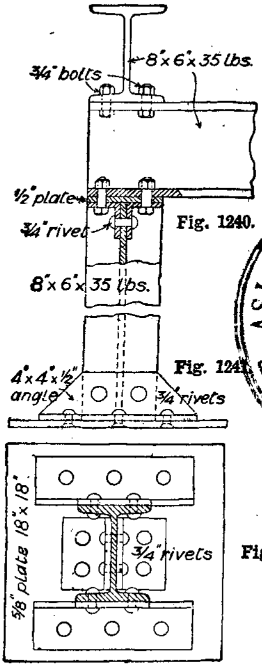

8"x6"x35 lbs.
3/4" bolts
1/2" plate
3/4" rivet
8"x6"x35 lbs.
4"x4"x1/2" angle
3/4" rivets
5/8" plate 18"x18"
3/4" rivets

Fig. 1240.

Fig. 1241.

Fig. 1242.

Figs. 1240 to 1242.—Details of Stanchion, etc.

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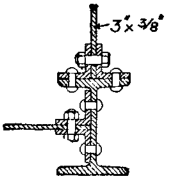


Fig. 1243.—Joint of Bracing Bar and Stanchion.

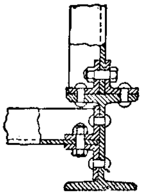
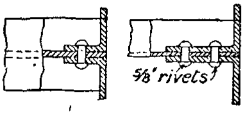


Fig. 1244.—Joint of Channel Bar and Stanchion.



Figs. 1245 and 1246.—Cleats on Ends of Rolled Joist and Channel-Beam.

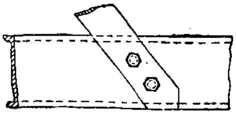


Fig. 1247.—Junction of Bracing Bar and Channel.

building 100 ft. diameter without internal supports, there is not much choice of method, and Figs. 1233 and 1234 show what is suggested. Fig. 1233 represents the section, and Fig. 1234 the plan. There are twenty half-trusses attached to and supported by two riveted angle and plate rings, 20 ft. diameter; this portion being surmounted by a conical roof with louvres round the upright portion. The main slopes of the roof can be covered with rough rolled plate glass supported upon the necessary purlins.

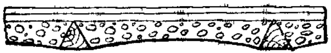


Fig. 1248.—Board of Works Fireproof Floor.

Fig. 1249.—Section of Skewback Joists cut from the Solid.

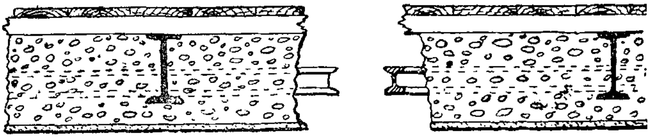


Fig. 1250.—Dawnay's Fireproof Floor.

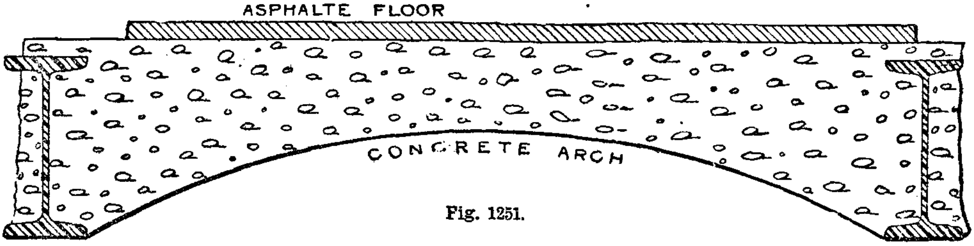


Fig. 1251.

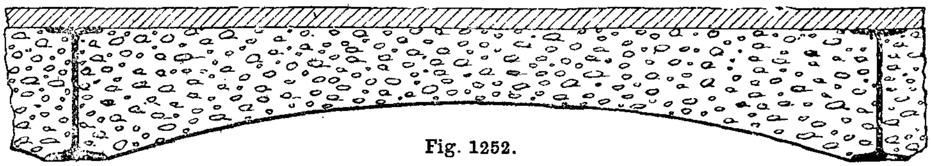


Fig. 1252.

Figs. 1251 and 1252.—Dennett's Fireproof Floor.

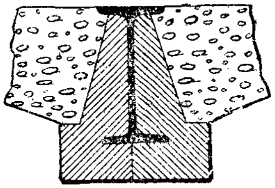


Fig. 1253.—Dennett's Improved Fireproof Floor.

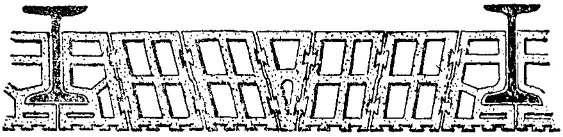


Fig. 1254.—The Doulton-Peto Fireproof Floor.

Plus and Minus Threads on Tie-Rod.

A plus thread is one standing out from the body of the rod or bolt, as shown in Fig. 1235, and a minus thread is one indented into the material, as shown in Fig. 1236. The strength of a tie-rod depending upon its least sectional area, the weakest part of a rod with the minus thread will be at the bottom of any of the threads, whereas with a plus thread the strength of the rod will be equal throughout. For instance, a tie-rod 1½ in. in diameter has a sectional area of 1·77 sq. in., which, at 5 tons per sq. in., equals a tensile strength of 8·85 tons. If the end of the rod be secured by a plus thread, no diminution of strength will take place, but if the end of the rod has a minus thread, the diameter at the bottom of the thread will be just a little more than 1¼ in., and the sectional area 1·3 sq. in., the corresponding tensile strength being 6·5 tons, or a loss on the previous case of 26½ per cent. Fracture is most likely to take place where the minus thread joins the full diameter of rod.

Structural Steelwork under Organ Chamber.

Fig. 1237 shows the front elevation, and Fig. 1238 a side elevation of the structure required to carry an organ chamber; Fig. 1239, plan of rolled joists at A; Fig. 1240, elevation at B, with part section; Figs. 1241 and 1242 elevation and plan of the lower end of the stanchion at C; Fig. 1243 horizontal section through the junction of the bracing bars and stanchion at D; Fig. 1244, horizontal section through the junction of the channel bars and stanchion at E; Fig. 1245, cleats on the end of the rolled joist at F; Fig. 1246, cleats on the end of the channel-beam at G; and Fig. 1247, the junction of bracing bar and channel at H.

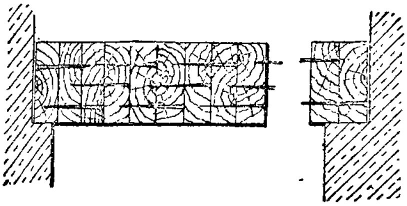
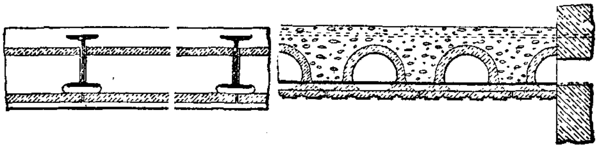


Fig. 1255.—Evans's Fireproof Floor.



Figs. 1256 and 1257.—Fawcett's Fireproof Floor.

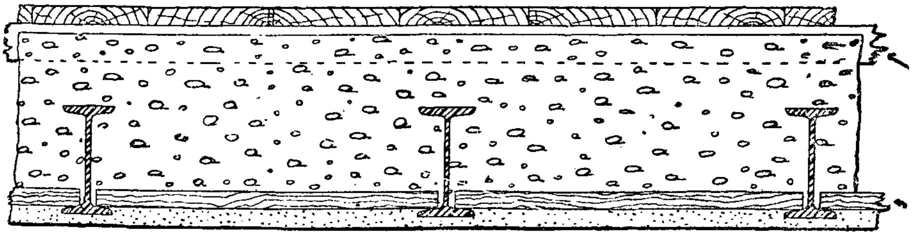
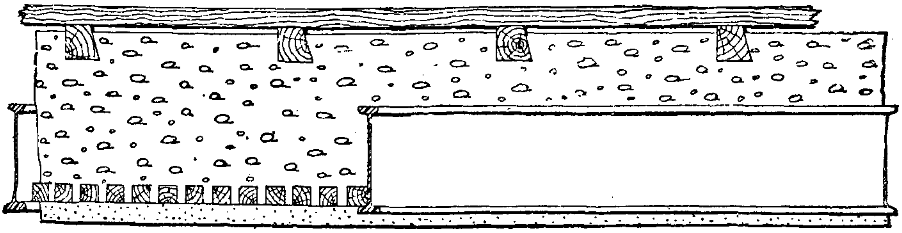


Fig. 1258.



Figs. 1258 and 1259.—Fox's and Barrett's Fireproof Floors.

Designing Concrete-Steel Floors for given Conditions.

No brief description can give the whole method of designing concrete-steel fireproof floors. The main girders must be calculated for their own strength only, but the cross girders, if embedded in suitable concrete, may be assumed to have an increased strength that may reach double the normal when the cross section has say 2 per cent. steel and the remainder concrete. A rolled joist without concrete cannot be used over a longer span than twenty times the depth of the joist, but this ratio may be increased to twenty-four times when the concrete is the same depth as the joist, to thirty times when the concrete is 50 per cent. deeper than the joist, and to thirty-six times when the concrete is twice the depth of the joist. A common formula for concrete-steel floors, when $\frac{1}{60}$ of the sectional area is steel in the lower half of the concrete, is $W = \frac{12t^2}{S^2}$, where S = span in feet, t = thickness in inches, W = safe load in cwt. per ft. super., but the concrete must be of the best 1 to 4 quality. Taking a thickness of 6 in. and a span of 16 ft., the safe load per ft. super. would be $W = \frac{12t^2}{S^2} = \frac{12 \times 6^2}{16^2} = 1{\cdot}69$ cwt.

Deducting ·55 cwt. for dead load will leave 1·14, say 1 cwt., external or live load. For a concrete steel beam in substitution for rolled joist as a support, the formula that is proposed by Professor Talbot, modified to suit the circumstances of the case, is as follows: $W = \frac{a\,t}{L^2}(570 - 2500\frac{a}{t}) - \frac{t}{10}$, where t = thickness of concrete in inches above the centre line of reinforcement, a = sectional area of reinforcement in square inches, w = weight of concrete in pounds per cubic foot, L = span in feet, W = safe external load in cwt. per foot run.

Fireproof Floors.

Board of Works.—Skewback joists of wood, with concrete arches of short span. Used in the offices of the late Metropolitan Board of Works, London, as in Figs. 1248 and 1249.

Dawnay's.—Rolled iron girders at intervals, with small iron joists or T-irons, bedded in concrete, left rough underneath for key to plastering, as in Fig. 1250.

Dennett's.—Iron girders with concrete arches between, composed of sulphate of lime and broken brick or pumice, as in Fig. 1251 or Fig. 1252. A later improvement is to protect the iron girders by terra-cotta blocks, as in Fig. 1253.

Doulton - Peto's.—Hollow stoneware blocks with cement keys placed between iron girders to form a flat arch, a specially moulded skewback block covering the girder, as in Fig. 1254.

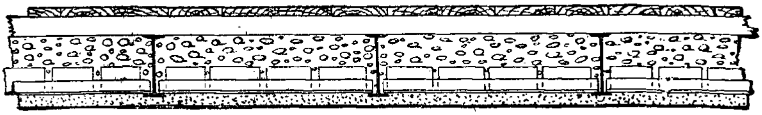


Fig. 1260.—Homan's Fireproof Floor.

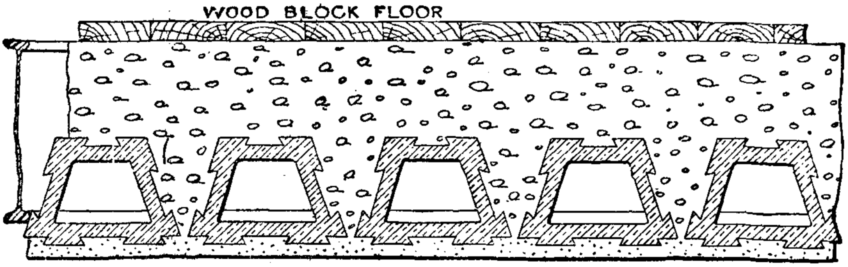
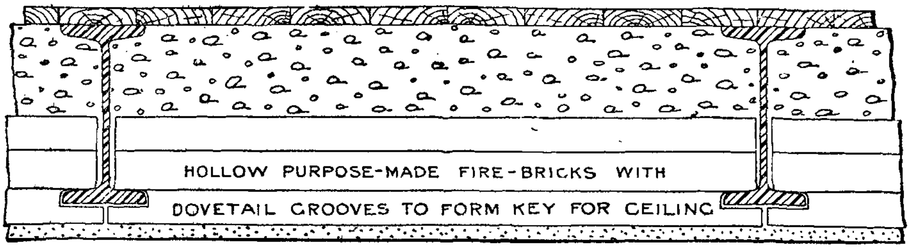


Fig. 1261.


HOLLOW PURPOSE-MADE FIRE-BRICKS WITH
DOVETAIL GROOVES TO FORM KEY FOR CEILING

Figs. 1261 and 1262.—Homan and Rodgers' Fireproof Floor.

Evans's.—Timber joists laid closely together, all cracks and shrinkages being grouted in with liquid plaster, as in Fig. 1255. Used in the Whitechapel warehouses of the East and West India Dock Co., London.

Fawcett's.—Rolled joists at intervals, earthenware tubular lintels being laid diagonally between them; the filling, as shown in Figs. 1256 and 1257, is coke-breeze concrete.

Fox and Barrett's.—Rolled joists at short intervals and rough wood fillets 1 in. to $1\frac{1}{2}$ in. square, half an inch apart, resting on lower flanges, space filled in with concrete above and plaster below, as in Figs. 1258 and 1259. Concrete to be composed of one part of Portland cement to five or six parts of coke breeze.

Homan's.—Rolled iron girders with joggled T-irons, upon which rest specially shaped hard burnt bricks with key grooves on underside for plaster, the whole covered with concrete, as in Fig. 1260.

Homan and Rodgers'.—Rolled joists at intervals; between and at right angles to them are hollow, purpose-made firebricks with dovetail grooves to form a key for the ceiling. Above the bricks is concrete flush with the tops of the rolled joists (see Figs. 1261 and 1262).

Hornblower's.—Iron girders, in fireclay tubes filled in with concrete, forming skewbacks for hollow fireclay blocks resting upon them, covered with concrete, as in Fig. 1263.

Lindsay's.—Rolled joists, with tie-rods over and under holding the concrete in place, as in Fig. 1264. Fig. 1265 shows the method of securing the rods by bending them over the flanges of the girders.

Lindsay's Trough System.—(See Statham's system below.)

Statham's.—Lindsay's steel trough sections, alternately inverted and riveted together, filled with concrete in which coke-breeze blocks or wood joists are bedded to nail flooring to, ceiling of plaster on wire lathing hung up to underside of troughs, as in Fig. 1266 or Fig. 1267.

Whichcord's.—Brick arches covered with concrete springing from fireclay moulded blocks encasing iron girders, as in Fig. 1268.

Wilkinson's.—Fibrous plaster pugging slabs, with fibrous plaster ceiling slabs on ordinary joists, as in Fig. 1269.

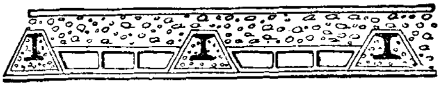


Fig. 1263.—Hornblower's Fireproof Floor.

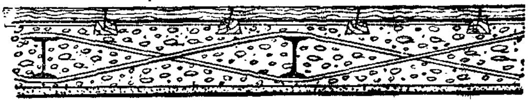


Fig. 1264.—Lindsay's Fireproof Floor.

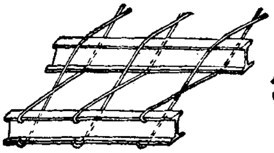


Fig 1265.—Securing Rods in Lindsay's Fireproof Floor.

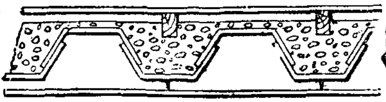


Fig. 1266.—Statham's Fireproof Floor.

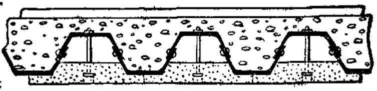


Fig. 1267.—Lindsay's Trough System.

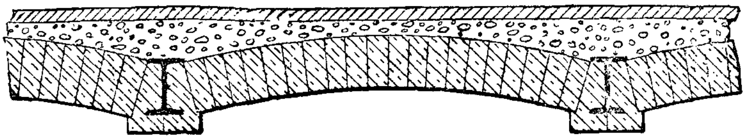


Fig. 1268.—Whichcord's Fireproof Floor.

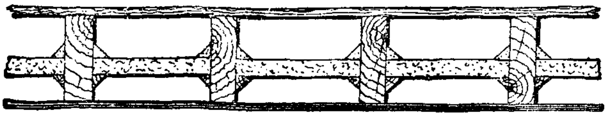


Fig. 1269.—Wilkinson's Fireproof Floor.

Fireproof Floor: Load, 2 Cwt. per Ft. super.—It is required to determine the necessary section for rolled iron floor joists at 8-ft. centres, the span being 22 ft., and making allowance for a possible load of 2 cwt. per ft. super., including the weight of the floor itself, which is to be fireproof, or more correctly fire-resisting. Area carried by each joist, 22 × 8 = 176 sq. ft.; load carried by each joist, 176 × 2 = 352 cwt.

Maximum bending moment on joist, $M = \frac{WL}{8} = \frac{352}{20} \times \frac{22}{8} = 48{\cdot}4$ ton-ft.

Depth of rolled joist not less than one-twentieth span, $\frac{22 \times 12}{20} = 13{\cdot}2$, say, 14 in.

Maximum stress in flange, $\frac{M}{d} = 48{\cdot}4 \div \frac{14}{12} = 41{\cdot}5$ tons.

Allowable tension, 5 tons per sq. in.

Sectional area required in flange $= \frac{41{\cdot}5}{5} =$ 8·3 sq. in.

Or, working another way, by the special formula for strength of rolled iron joist, $W = 7\,\frac{a\,d}{L}$,

where W = breaking weight tons centre,

a = area bottom flange plus one-sixth web,

d = depth in inches. L = span in ft.

Whence, by transposition, $a = \frac{W\,L}{7\,d} \div 2$ for distributed load, and × 4 for factor of safety,

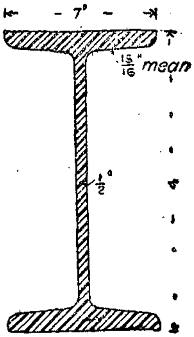


Fig. 1270.—Section of Rolled Joist.

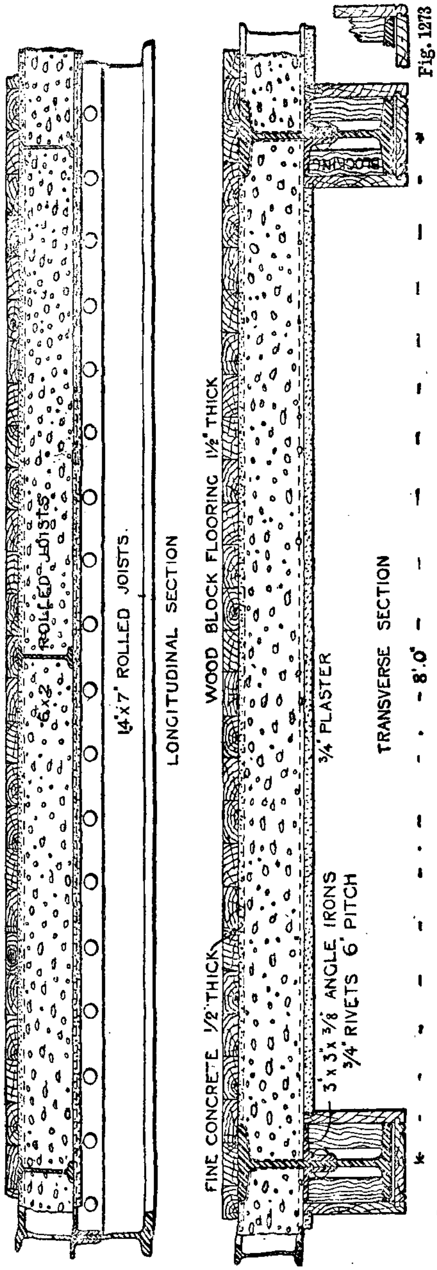
then area $= \frac{4 \times W\,L}{2 \times 7\,d} = \frac{0{\cdot}3\,W\,L}{d}\,;\ \frac{0{\cdot}3 \times 17{\cdot}6 \times 22}{14}$ = 8·3 sq. in. as before.

In the sectional area of flange, one-sixth of the depth of web is included, and this will generally equal about one-fourth of the actual flange area, so that the actual flange area should be four-fifths of the area found above, 8·3 × $\frac{4}{5}$ = 6·6 sq. in., and the whole section would be as Fig. 1270. To construct the floor, use intermediate joints 6 in. by 2 in. at 4 ft. centre to centre; support these on 3-in. by 3-in. by ⅜-in. angle-irons riveted to web of main joists; fill in concrete to a uniform depth of 6 in. over the intervening spaces, and cover with wood-block flooring, as in Figs. 1271 and 1272, which show longitudinal and transverse sections. Fig. 1273 shows alternative arrangement of wood casing to main girders. If preferred, terra-cotta or fibrous plaster protecting blocks may be used.

Floor for Public Building.—The floor of a public building must be strong enough to bear, besides its own weight, a safe live load of 1½ cwt. per sq. ft. Assuming that a floor is built of rolled steel joists and concrete, the weight for a span of 25 ft. will be about ¾ cwt. per sq. ft., making a total of 2¼ cwt. With a span of 25 ft. the depth should not be less than 14 in., and if the joists are made 14 in. by 6 in. by 57 lb. per ft. run, they will carry 16 tons each distributed. This means a centre distance of $\frac{16 \times 20}{25 \times 2\frac{1}{4}}$ = say, 5 ft. 8 in., but the main girders should be arranged so as to come in the piers between the windows. If the girders are placed farther apart than stated above, they should be deeper, or two girders should be used side by side. With a centre distance of 5 ft. 8 in. cross girders are not necessary, provided the concrete is not less than 6 in. thick and is strengthened by expanded metal at the bottom; but if the distance exceeds 6 ft., cross girders, with a depth in inches equal to half the span in feet, must be provided, as well as the concrete and the expanded metal. A total live and dead load of 2 cwt. per sq. ft. should be provided for in the floor of a public concert hall.

Concrete Steel Floor for School.—For a school hall, the external load provided for should be 1½ cwt. per ft. super., and the floor itself, if the concrete be 10 in. thick, will be another 1 cwt., making 2½ cwt. gross load per ft. super. With a span of 23 ft. 6 in. for the main girders, and 10 ft. centre to centre, the load on each will be $\frac{23{\cdot}5 \times 10 \times 2{\cdot}5}{20}$ = say 30 tons, for which a 20-in. by 7½-in. by 89-lb. rolled steel joist would be just sufficient. With 4-in. by 2-in. by 8-lb. rolled steel joists, 20 in. centre to centre, at the lower side of 10-in. thickness of coke-breeze concrete 1 to 6, the safe gross load would be about 3 cwt. per ft. super., providing the concrete is put in by specialists; if it is put in by ordinary workmen, the proportion of cement should be increased, and close supervision should be exercised. The angle-iron supporting the small joists may be 3 in. by 3 in. by ⅜ in., fixed with ¾-in. rivets, 4-in. pitch. Every fourth cross joist should be attached to the main girder by a standard angle bracket.

Concrete Steel Floor of 15-ft. 6-in. Span.—The steel beams of a floor are to be of 15-ft. 6-in. span, and the load 50 tons. If four rolled joists are to be used, they should be placed as in Fig. 1274, where the loading will be equal, and not as in Fig. 1275, where the joists next to the wall will only have half the load upon them that the others have. As the beams are 15-ft. 6-in. span, they should not be less than 8 in. deep, but may need to be more; and as the total load is 50 tons, each will have to carry 12½ tons. To carry 12½ tons over 15-ft. 6-in. span, 8-in. by 6-in. by 35-lb. rolled steel joists may be used with a factor of safety of 3, but it would be more satisfactory to have them 10-in. by 6-in. by 45-lb. with a factor of safety of 4½, or 12-in. by 5-in. by 39-lb. with the same factor of safety. A load of 50 tons on an area of 20 × 15·5 = 310 sq. ft. will give $\frac{50 \times 20}{310} = 3{\cdot}226$, say 3¼ cwt. per ft. super., $w = c \times \frac{t^2}{L^2}$, or $t^2 = \frac{w L^2}{c} = \frac{3\frac{1}{4} \times 5^2}{3\frac{1}{2}} = 23{\cdot}3$, for 1 cement to 5 aggregate, whence $t = \sqrt{23{\cdot}3}$ = say 5 in. if laid by specially experienced men, otherwise 20 per cent. extra at least must be allowed, making the minimum thickness 6 in. The floor would be materially strengthened by the use of expanded metal either on the bottom, or 1 in. above the bottom, of the concrete, laid continuously with overlap across the joists. If expansion joints are left in the concrete to grout up afterwards, they should be over the centre of the joists.



Figs. 1271 and 1272.—Longitudinal and Cross Sections of Fireproof Floor.

Fig. 1273.—Wood Casing to Main Girders.

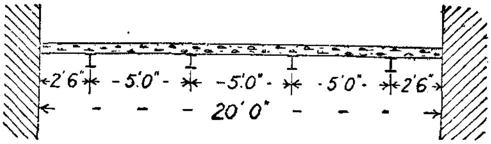


Fig. 1274.—Concrete Steel Floor Loaded Equally.

Fireproof Floor for Offices.—A single system of rolled joists would probably not be so economical across a span of 17 ft. as main girders with intermediate cross joists; but the plan can be tried. Say 8-in. by 4-in. by 25-lb. rolled steel joists, 3·5 ft. centres, concrete 8 in. deep, weighing 80 lb. per ft. super., external load 1¼ cwt. = 140 lb., or total of 80 + 140 + 7 for rolled joists = 227 lb. per ft. super. At 3·5 ft. centres and 17-ft. span, the total load on

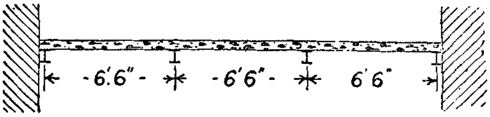
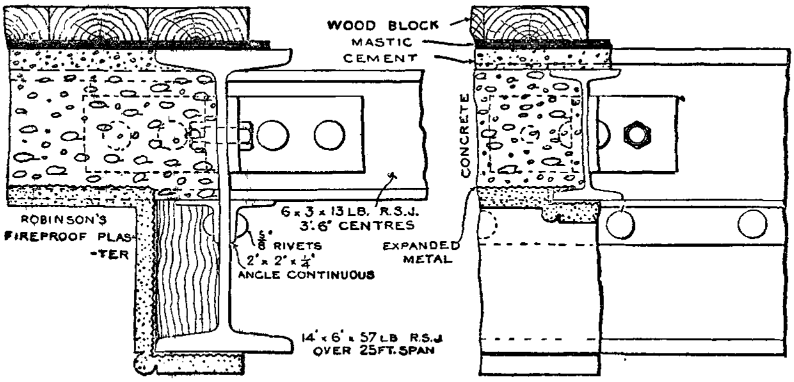


Fig. 1275.—Concrete Steel Floor Loaded Unequally.

each joist will be 227 × 3·5 × 17 ÷ 2240 = 6 tons. The maximum safe load on an 8-in. by 4-in. by 25-lb. rolled steel joist at 17-ft. span, if free $= \frac{97\cdot65 \text{ tabular value}}{17} = 5\cdot74$ tons; but by Wilkinson's rule for concrete steel floor, W = $32\frac{I}{L^2}$, where W = safe distributed load in tons, I = moment of inertia of section in inch-tons, L = span in feet. Then $W = 32 + \frac{73\cdot24}{17^2} = 8\ 11$, say 8 tons, which may be considered suitable as providing a little extra margin against the risk of the concrete not being put in by specially skilled men. If, on the other hand, a compound system of joists be adopted, the main girders would be placed in the solid piers between the windows, so that the number would depend on the construction of the building. The girders should not be less than 9 in. deep, and the section that is required will depend on the load that has to be carried, which again will depend on the distance apart that the girders are placed. The intermediate joists may then be 2 ft. 6 in. or 3 ft. apart, and possibly as small as 4 in. deep, with concrete 6 in. deep, but all this will depend on the position of the main girders.


WOOD BLOCK
MASTIC
CEMENT
CONCRETE
ROBINSON'S FIREPROOF PLASTER
6 x 3 x 13 LB. R.S.J. 3'.6" CENTRES
5/8" RIVETS
2" x 2" x 1/4" ANGLE CONTINUOUS
EXPANDED METAL
14" x 6" x 57 LB R.S.J OVER 25FT. SPAN

Figs. 1276 and 1277.—Connection of Joists in Fireproof Floor.

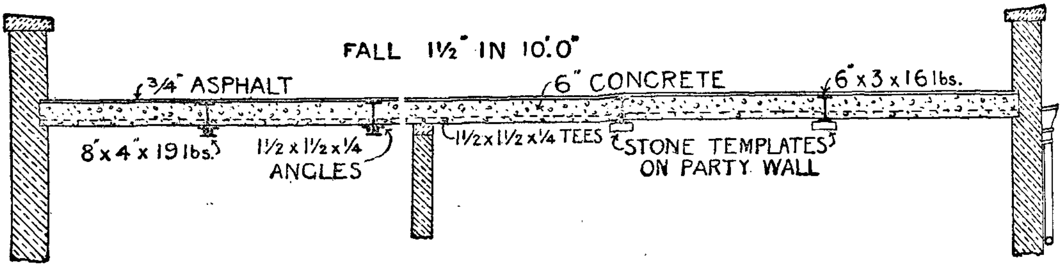

FALL 1½" IN 10'.0"
3/4" ASPHALT
6" CONCRETE
6"x3x16 lbs.
8"x4"x19 lbs.
1½x1½x¼ ANGLES
1½x1½x¼ TEES
STONE TEMPLATES ON PARTY WALL

Fig. 1278.—Section of Concrete Steel Roof.

Concrete Floors Stiffened with Iron Rods and Expanded Metal.—Experiments on the strength of concrete floors stiffened with ¼-in. iron rods or expanded metal show that, where the load and span are suitable, these methods are more economical than the use of small rolled joists. A small joist embedded in concrete adds considerably to the strength, and when contained entirely in the lower half of the thickness, may be reckoned practically as double in value. A simple concrete floor, 1 cement to 4 ballast, if well laid over a span in feet equal to the thickness of concrete in inches, may at the end of a month be considered safe with a load of 2 cwt. per ft. super., including the weight of the floor.

Radius of Gyration.—The radius of gyration for a rolled joist can only be obtained accurately by taking the exact section. The formula is, $r^2 = \frac{I}{A}$, where r = radius of gyration, I = moment of inertia, A = sectional area. Taking a 12-in. by 6-in. by 54-lb. rolled steel joist with ½-in. thickness of web and ⅞-in. flanges, the moment of inertia is 369·91, and the sectional area 15·9 sq. in.; the radius of gyration is therefore $\sqrt{\frac{I}{A}} = \sqrt{\frac{369{\cdot}91}{15{\cdot}9}} = \sqrt{23{\cdot}26} = 4{\cdot}8$ in.

Concrete Floor for Kitchen.—For a concrete kitchen floor, 14-ft. span, over cellars, the smallest rolled steel joists embedded in cement concrete will be 6 in. by 3 in. by 13 lb. per ft. run, placed 2 ft. 6 in. to 2 ft. 9 in. centre to centre, with Portland cement concrete (1 cement to 4 ballast) level with the underside of the joists and reaching 1 in. above the joists, or 7 in. in all, the top inch being fine concrete floated over with neat cement.

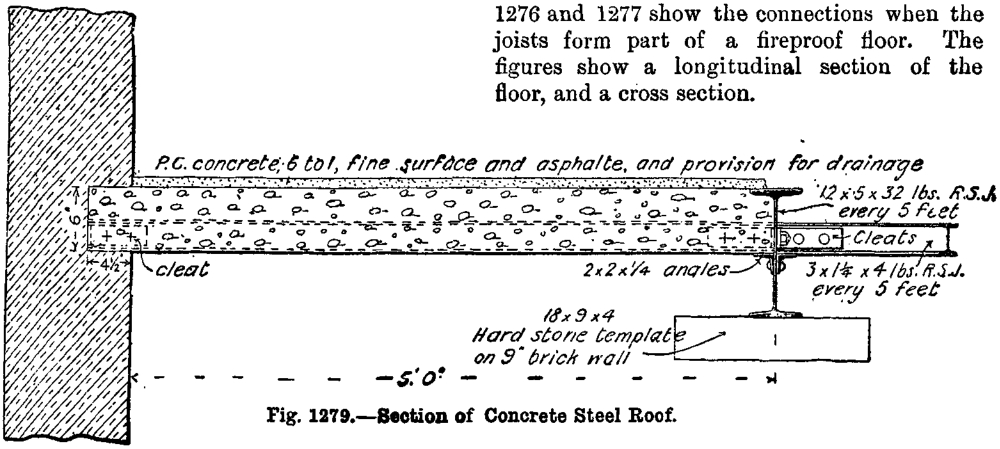

P.C. concrete, 6 to 1, fine surface and asphalte, and provision for drainage
12 x 5 x 32 lbs. R.S.J. every 5 feet
6"
4½
cleat
Cleats
2 x 2 x ¼ angles
3 x 1¼ x 4 lbs. R.S.J. every 5 feet
18 x 9 x 4 Hard stone template on 9" brick wall
5' 0"

Fig. 1279.—Section of Concrete Steel Roof.

Concrete Upper Floor for Stable.—It is proposed, in a building 18 ft. clear in width between walls, and 70 ft. long, to stable horses on the first floor. The floor to be composed of Young's patent paving bricks, over rolled steel joists bedded in concrete. Rolled joists used for a span exceeding twenty times their depth require careful packing when they carry concrete, otherwise the deflection will be very liable to cause fracture. A horse weighs from 9 cwt. to 18 cwt.; say, therefore, 1 cwt. per ft. super. to allow for vibration of moving load. Then, with 12 in. thickness of concrete, and 3 in. thickness of paving blocks, the dead load will be 1¼ cwt. per ft. super., or a total load of, say, 2½ cwt. per ft. super., including joists and external load. For this purpose a section not less than Dorman, Long and Co.'s 10-in. by 5-in. by 29-lb. rolled steel joists should be used at 3-ft. centres. Cross joists will not be necessary if the concrete is of good quality, but in case of fire it will be an advantage to have T-irons 3 in. by 3 in. by ⅜ in., web upwards, laid on the bottom flange of joists 3 ft. centre to centre.

Connection of Joists in Fireproof Floor.

Joist connections by means of angle plates have already been illustrated, and now Figs. 1276 and 1277 show the connections when the joists form part of a fireproof floor. The figures show a longitudinal section of the floor, and a cross section.

Concrete Steel Roofs.

It is required to know the number and the size of the iron joists required to support a flat roof of coke breeze and cement, the roof to be used for drying purposes; the external walls are one brick thick, and the partitions half a brick thick. The roof is over a front room and two back rooms. For the front room (10 ft. by 15 ft.) two rolled steel joists, each of 8 in. by 4 in. by 19 lb. per ft. run, would be required to carry the roof over the 15-ft. span; and for the two back rooms (each 12 ft. by 9 ft. 6 in.) two rolled steel joists, each of 6 in. by 3 in. by 16 lb. per ft. run, and each with a span of 19 ft. 6 in., would carry the roof over the two rooms, but the joists must be supported at their centres by stone templates on the brick partition that divides the rooms. The whole arrangement is shown in Fig. 1278. The deep joists should have 1½-in. by 1½-in. by ¼-in. angles on each side to carry the concrete, and it may be found desirable to put T-section beams 1½ in. by 1½ in. by ¼ in. every 3 ft. as a further support to the concrete.

Another Case.—Assume that it is intended to construct a flat roof of steel joists, concrete, and asphalt strong enough to carry a crowd of people, with a good factor of safety, without having supporting columns in the rooms below, and with as little steelwork as possible. A crowd of people will weigh 100 lb. per ft. super. A steel joist floor and 6-in. concrete will weigh about 60 lb. to 70 lb. per ft. super. The whole load to be allowed for will therefore be 1½ cwt. per ft. super. The span being 22 ft., the least section of main rolled steel joists should be, say, Dorman, Long & Co., G 7a, 12 in. by 5 in. by 32 lb. per ft. run, which will carry a distributed load of 8¼ tons. To make up 8¼ tons, 110 sq. ft. area must be supported; this at 22-ft. span will be a maximum width of $\frac{110}{22} = 5$ ft., or say four rows of joists dividing the width of 25 ft. into five parts, and allowing for the support of the front and back walls to the outer edges of the concrete, as per section shown in Fig. 1279.

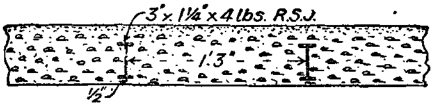


Fig. 1280.—Joists in Concrete Steel Roof at 15-in. Centres.

Concrete Roof to Back Addition.—The back addition of a six-room cottage is to have a flat concrete roof, 10 ft. square, resting on steel joists, the underside to form the ceiling. A channel may be formed near the lower end of the flat, to lead the water to the down pipe, but it would perhaps be cheaper and more efficient to put an iron gutter along the lower edge, with an outlet into a rain-water head. Assume 3-in. by 1¼ in. by 4 lb. per ft. run rolled steel joists, the unsupported strength on a 10-ft. span will be $\frac{5{\cdot}22}{10} = {\cdot}522$ ton distributed. With the addition of good cement concrete 1 to 4, the strength will be raised to ·522 × 2 = 1·044 tons, say 1 ton distributed. Including weight of concrete, allow 1½ cwt. per ft. super., as load to be carried, then $\frac{1 \times 20}{1\frac{1}{2} \times 10} = \frac{20}{15} = 1\frac{1}{3} = 1$ ft. 4 in. centre to centre for the joists, but 1 ft. 3 in. will work in better, and may be adopted as in Fig. 1280. The detail at lower and upper edges will then be as in Fig. 1281. The general arrangement will be as Fig. 1282 in elevation and Fig. 1283 in plan. This would, of course, be a substantial roof, and suitable for use as a drying ground.

Treatment of Joists Embedded in Concrete

Iron or steel joists that are to be embedded in concrete for upper floors need not be painted with oil paint; but, after removing any rust with a wire brush, paint the joists over with a Portland cement wash. Joists embedded in concrete for foundations are usually put in with only the manufacturer's one coat of paint on; nevertheless, such joists may, after cleaning, receive a coat of boiled linseed oil.

Concrete Blocks and Artificial Stone.

The term concrete blocks is generally limited to blocks that contain a fairly large aggregate such as shingle; these blocks are used for building river and sea walls, groynes, breakwaters, etc. The term artificial stone is applied to blocks that contain small aggregate, such as granite chippings; these blocks are moulded into finished forms such as paving slabs, curbs, steps, lintels, etc. Both kinds of stone have been in use for many years

Materials for Concrete.

Broken stone should be free from dust and dirt, hard, rough, angular, and capable of passing a 2-in. ring. It is one of the best possible aggregates when of good quality.

Burnt clay must be thoroughly burnt and semi-vitrified, and large pieces broken to pass a 2-in. ring. All partially burnt pieces must be rejected. It may be used for lime concrete as an aggregate where no heavy loads are to be carried, but is not strong enough for the best cement concrete. Might be used in a mortar mill in place of sand where sand is scarce.

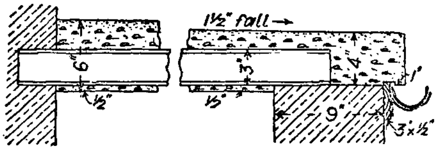


Fig. 1281.—Section of Concrete Steel Roof.

Furnace slag must be free from uncombined lime and not too glassy. It is better when obtained in large pièces and broken to pass a 2-in. ring. Suitable material of this kind makes the strongest possible cement concrete, but is rather heavy.

Hard chalk.—This is sometimes used in its natural state rammed hard in two layers of 4 in. each, as a bed or foundation for stacking ground for coals, and is also used in lumps as a backing to retaining walls to prevent water collecting. It is not used for concrete unless burnt into lime. For this purpose it should consist chiefly of carbonate of lime with 10 to 20 per cent. of alumina. Any silica in it reduces its strength as a cementing material.

Pit gravel should be free from loam or earthy admixture. A proportion of sand is advantageous, and large pieces should be broken to pass a 2-in. ring. The more the gravel tends to bind when laid on a road or footpath the less suitable it would probably be for concrete, owing to an undue amount of loam in the sand.

River sand must be free from mud or ooze and as large grained as possible. A bright sparkling stream is more likely to furnish it than a sluggish one.

Shingle should be free from mud and seaweed, but may have a large admixture of sand. Shingle from the seashore if containing much sand, would not be suitable for lime concrete, owing to the attraction it would have for the moisture of the air. The large shingle, broken to pass a 1½-in. ring, is a good addition to common shingle on account of its angularity.

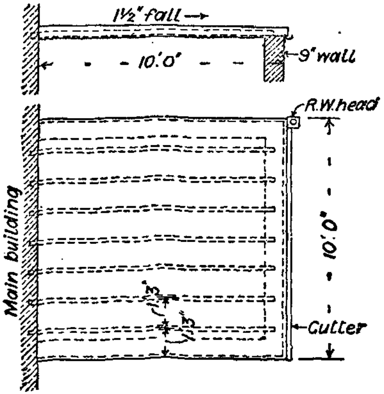
Smiths' clinkers are similar to furnace slag, but not so hard. They may be broken up and ground in a mortar mill instead of sand.

Thames ballast consists chiefly of rounded flint pebbles of various sizes, with some sand and much mud; but when "washed" only pebbles are left. These are very hard, and suitable in that respect, although, being rounded, they cannot bond at all, and are therefore unsuitable for transmitting pressure, and they are also non-porous, so that there is some difficulty in getting sufficient adhesion to the matrix.

Hard stone, broken to pass a 2-in. or 2½-in. ring or mesh, is not open to any of these objections, and would be better than Thames ballast when it could be obtained in sufficient quantity. A proportion of Thames ballast, not exceeding one-half, might be mixed with it for economy.

Principles of Concrete Mixing.

Concrete consists of two main parts, the aggregate of hard lumps, and the matrix in which they are embedded. The matrix should be composed of the best materials available, to form a mortar of the strongest description. Portland cement will be the best cementing material; slow setting if the concrete is to be used in large masses. It should weigh about 112 lb. per bushel, pass through a mesh 2,500 to the square inch, with not more than 2½ per cent. residue, and be thoroughly air-slaked under cover. A briquette after immersion in water for seven days should bear a tensile stress of not less than 350 lb. per square inch. Either pit sand or sea sand, if clean and sharp, free from loam, mud, or earthy matter, will be equally suitable for mixing with the cement. The proportions for the matrix should be such that the mortar formed should be sufficient in quantity to fill thoroughly the voids of the aggregate, with a little to spare in case of imperfect mixing. The contents of the voids of the aggregate may be ascertained by weighing a cubic foot of the solid stone and of the broken stone, and noting the difference. If the aggregate is stone broken to a 2-in. gauge, 1 cubic yd. will contain about 12 cubic ft. of voids; and if to every cubic yard of the aggregate be added 5 cubic ft. of Portland cement and 10 cubic ft. of sand, which makes about 13 cubic ft. of Portland cement mortar, a strong



Figs. 1282 and 1283.—Elevation and Plan of Concrete Steel Roof.

concrete will be formed, with sufficient mortar to fill the voids and a little more to allow for waste, etc. This will give 1 cement, 2 sand, 5 stone. The materials should be thoroughly mixed dry, then watered through a rose and mixed wet, deposited as soon after mixing as possible in layers not exceeding 12 in. thick, and rammed to fill the voids. In ramming, care must be taken not to disturb the concrete that has already set. It will be noted that cement is ground finer and has more strength than was formerly the case.

Specification for Concrete for Thin Arches.

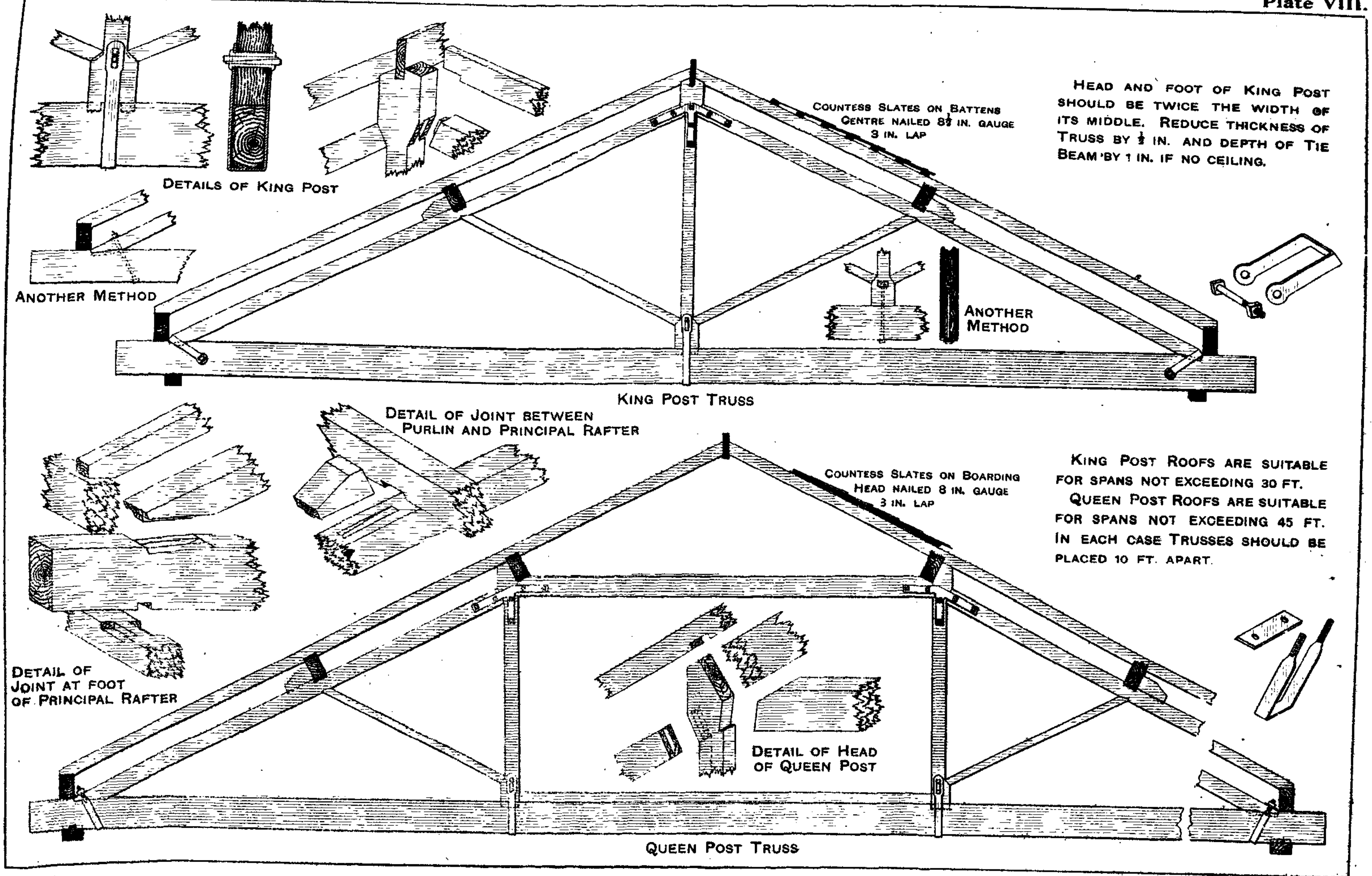
The concrete to be composed of 1 part by measure of Portland cement to 4 parts of coke breeze. The cement to be best air-slaked Portland cement, weighing not more than 110 lb. per imperial striked bushel, and 95 per cent. to pass through a sieve of 2,500 meshes per square inch ; briquettes, after seven days (six in water), to break with an average of not less than 325 lb. per square inch, and a minimum of 300 lb. per square inch. The coke breeze to be broken to pass entirely through a ¾-in. mesh and to be screened free from dust. The materials to be carefully mixed dry in small quantities, and thoroughly re-mixed while being watered through a rose, before depositing.

Specification for Concrete for Retaining Wall.

The concrete to be composed of 1 part by measure of Portland cement, 2 parts sand, and 7 parts ballast. The Portland cement to be of best quality, air-slaked, weighing not less than 115 lb. per imperial striked bushel, 90 per cent. to pass through a sieve of 2,500 meshes per square inch; briquettes, after seven days (six in water), to sustain not less than 350 lb. per square inch without fracture, and after twenty-eight days not less than 450 lb. per square inch. The sand to be clean and sharp, free from all earthy admixture. The ballast to be free from loam, earth, or dirt of any kind, and broken to pass a 2½-in. ring, and may consist of gravel, flints, hard stone chippings, burrs, clinkers, glass or ironworks refuse, or other material, provided it be hard, free from dust or dirt, and pass the 2½-in. ring. The concrete to be mixed dry in small quantities, turned well over, watered through a rose, and again thoroughly mixed, wheeled at once to the trenches, and tipped. To be levelled at time of tipping in layers 12 in. thick, and not to be again disturbed.

Specification for Concrete Foundations and Walls of Dwelling-House.

The following is a specification for the concrete foundations and walling of a dwelling-house ; the walls to be built in coursed rubble with cut stone quoins, and an inner lining of 4½-in. brickwork. *Excavator and Concreter.*—Excavate the trenches for all walls to the depths and widths shown or figured on the drawing, or to such additional depth as may be found necessary to secure a firm substratum for building upon. Lay under all walls a bed of concrete, composed of 1 part of best fresh burnt Dorking, Halling, or Merstham ground stone lime to 2 parts of clean sharp fresh-water or pit sand, and 5 parts of clean gravel or broken bricks in pieces that will pass through a 2 in. ring. The materials for the concrete to be measured and mixed dry on a wooden platform, to prevent admixture of earth or other objectionable matter, moistened with water through a rose, and turned over twice before being deposited. The concrete to be carefully spread and levelled, and left to set before the walls are proceeded with. *Stonemason* (*or Waller*).—Construct the walls throughout of the dimensions shown or figured on the drawings, with straight perpendicular faces of hammer-dressed coursed Kentish rag rubble (or of rubble stone found in the locality and brought up to courses, and made level at every 3 ft. throughout the entire height). Lay the footings in large single stones of equal thickness upon the concrete above described, and of a projection on each side equal to half the thickness of the wall. Lay a damp-proof course of two courses of slates, breaking joint, in Portland cement mixed with an equal quantity of sharp sand, immediately above ground level (say 3 in. above) through the whole thickness of the wall. Build in hoop-iron holdfasts, with ends turned up and down, No. 16 B.W.G., and tarred and sanded, one every 2 ft. in length and every 1 ft. 6 in. in height. The Kentish rag is to be free from hassock, and to be laid on its natural bed, and great care is to be taken to avoid iron


DETAILS OF KING POST
ANOTHER METHOD
COUNTESS SLATES ON BATTENS CENTRE NAILED 8½ IN. GAUGE 3 IN. LAP
ANOTHER METHOD
KING POST TRUSS
DETAIL OF JOINT BETWEEN PURLIN AND PRINCIPAL RAFTER
COUNTESS SLATES ON BOARDING HEAD NAILED 8 IN. GAUGE 3 IN. LAP
DETAIL OF JOINT AT FOOT OF PRINCIPAL RAFTER
DETAIL OF HEAD OF QUEEN POST
QUEEN POST TRUSS

HEAD AND FOOT OF KING POST SHOULD BE TWICE THE WIDTH OF ITS MIDDLE. REDUCE THICKNESS OF TRUSS BY ½ IN. AND DEPTH OF TIE BEAM BY 1 IN. IF NO CEILING.

KING POST ROOFS ARE SUITABLE FOR SPANS NOT EXCEEDING 30 FT.

QUEEN POST ROOFS ARE SUITABLE FOR SPANS NOT EXCEEDING 45 FT. IN EACH CASE TRUSSES SHOULD BE PLACED 10 FT. APART.

FRAMING OF SMALL ROOFS.

stains by rejecting for face work all stones having iron pockets on face. The courses are not to be less than 9 in. nor more than 12 in. high, and to be carefully bonded, well flushed up with mortar and grouted every course. The facing is to average 8 in. on bed, and to have at least one bond stone to every yard super. Rake out and point at completion with a neat struck mason's joint of Portland cement and sand in equal proportions. The mortar is to be composed of one part best Dorking, Halling, or Merstham ground stone lime to two of clean sharp freshwater or pit sand, and mixed in small quantities. The angles of the walls to be formed with cut and dragged Box Ground Bath stone quoins, free from cracks, stains, or other imperfections, to show alternately 9 in. and 18 in. on face and return, and to hold full dimensions to back. Box Ground Bath stone, sunk, weathered, and throated sills, and lintels with rubble relieving arches over, to be put to all window openings, and hard York stone rubbed and cleansed steps to all external doorways. Attend upon and make good after other trades, and leave clean and perfect on final completion. *Bricklayer.*—Build in cement a 4½-in. backing of sound, hard grey stocks left rough for plastering, on the stone foundation prepared by the mason, with hoop-iron bond No. 16 B.W.G., one tier to every 18 in. in height, and tarred and sanded, carried to the full height of the stonework, and with corresponding openings. Attach the ends of the iron holdfasts built into the stonework by the mason in this hoop-iron bond. Lay a damp-proof course in the same manner and at same height as in stonework. Build in as the work proceeds all wood plugs, wall plates, and ends of joists; pin ends, etc., and do all other necessary work. Render, float, and set the whole of the internal face of the brickwork, and twice colour same to approved tint. Attend upon and make good after other trades, and leave all clean and perfect at the final completion.

Specification for Cement and Concrete for Floor.

The following specification governs the supply of the cement, and the making and laying of the concrete, for a floor to be carried by rolled steel joists. The Portland cement used in the construction of the fireproof floor is to weigh not more than 110 lb. per imperial striked bushel, and to pass through a sieve of 2,500 meshes per square inch, with a residue not exceeding 2½ per cent. of the whole. The cement to be delivered at least fourteen days before required for use, spread out at once on a dry floor, and turned over thoroughly four times at intervals of not less than three days. Briquettes with a net sectional area of not less than 1 sq. in. are to be gauged with neat cement and left twenty-four hours in air, then placed for six days in water, and must break with an average on every three consecutive specimens of not less than 350 lb. per square inch tensile stress. The concrete is to be composed of 1 part cement, 2 sharp sand or small sandy gravel, and 5 hard clean broken brick passing a 1-in. ring (or 1 cement, 3 coke breeze, 3 broken brick; or 1 cement, 6 coke breeze). The broken brick is to be well wetted with clean water before being mixed with the sand and cement. The materials are then to be separately measured in specially made boxes and thoroughly mixed dry in small quantities on a wood platform, and again turned over twice while sufficient water is added from a rose to incorporate them thoroughly without washing away any cement. Any concrete mixed more than one hour before use will be condemned as unfit, and must forthwith be removed from the premises. The underside of the floor is to be closely boarded and strutted to support the concrete, and left until the concrete has thoroughly set. The concrete is to be put in in one layer and closely packed between the flanges of the rolled joists, and finished off smooth with the spade on top. The flooring may be composed of Portland cement mortar finished level and true 1 in. thick, in the proportion of 1 cement to 3 sand, and wood-block flooring bedded on mastic. The underside of floor finished with plaster to form ceiling in the usual way.

Strength of Concrete.

Mr. Grant, of the late Metropolitan Board of Works, found that the strength of concrete regularly diminished as the proportion of cement became less. Approximately the results follow the formula $F = 150 - 10B$, where F = crushing force in tons per square foot and B = quantity of ballast to 1 of cement. Sutcliffe's "Concrete" quotes three tests by

Kirkaldy for strength of concrete beams as follows:—(1) Beam of 1 Portland cement and 1 coke breeze, seven days old, 3 in. broad, 5 in. deep, 72-in. clear span. Breaking weight loaded in centre averaged 3·85 cwt., or allowing half-weight of beam between supports a gross central load of 4·07 cwt. (2) Beam of 1 Portland cement and 2 crushed bricks, two or three months old, 12 in. broad, 8 in. deep, 60-in. span. Breaking weight loaded in centre averaged 13·25 cwt, or a gross central load of 15·08 cwt. (3) Beam of 1 Portland cement to 6 gravel, ninety days old, 12-in. by 12-in. by 36-in. span. Average breaking weight on central 6 in. = 46·67 cwt. From the writer's observation, the strength is subject to so many contingencies that experiments cannot be relied upon very closely. A reasonable practice is to let the thickness of concrete in inches equal the span in feet between main joists, and to put cross joists of about half-depth at half the span apart.

Strength of Concrete Lintels.

A concrete lintel 8 in. wide and 6 in. deep, the concrete being composed of 1 Portland cement, 2 sand, 4 broken stone, one month old, would be calculated by the formula $W = \frac{c\,b\,d^2}{L}$, where W = breaking weight in the centre in hundredweights, including half the weight of the lintel, c for 1 to 4 = ·2, 1 to 5 = ·14, 1 to 6 = ·06, b d = breadth and depth in inches, L = span in feet. Assuming the span to be 4 ft., then $W = \frac{\cdot05 \times 8 \times 6 \times 6}{4} = 4\cdot32$ cwt. breaking weight in centre. With a uniformly distributed load, and a factor of safety of 4, the gross working load would be 2·16 cwt. Taking its own weight as $1\frac{1}{3}$ cwt., this would allow for a maximum external load of only 1 cwt. if one month old, 2 cwt. if three months old, and 3 cwt. at six months old.

Safe Thickness of Coke-Breeze Concrete Arches.

The safe thickness of a coke-breeze concrete arch does not depend on the span alone, but on the strength of the concrete, the rise, span, and shape of the arch, and the nature, amount, and distribution of the load. Assuming an arch of the proportion shown in Fig. 1284, and 1 to 4 coke-breeze concrete made and laid by specialists, the safe load at the end of one month would be 16 cwt. per ft. super. If this was mixed and laid by ordinary labourers, with ordinary supervision, the safe load would probably be reduced to 5 cwt. per ft. super., which would then be strong enough for an ordinary warehouse. The weight of the concrete itself should be reckoned as part of the load. In concrete construction, very much depends on the skilled labour employed.

Moisture and Condensation on Concrete Walls.

The reason why concrete houses show damp inside when a rise of temperature occurs is not because damp comes through the walls, but for exactly the same reason that the surface of a varnished hall-paper (in a small house) shows moisture on "washing day." As the temperature of air rises, its capacity for absorbing moisture increases, and a drop in the temperature has an opposite effect; therefore when the temperature of the air rises suddenly, the cold walls cool the adjacent air and its moisture is deposited on the walls. Plastered walls and plain papered walls do not show the effect of a change of temperature, because the moisture soaks into the wall as fast as deposited but a varnished cupboard in the room will show the damp deposit. The concrete walls might perhaps be plastered and thus remove the annoyance; but the difficulty is to get a good key for the plaster.

Provision for Expansion and Contraction.

In forming concrete arches, or concrete paving in position, expansion boards or strips of wood are laid in, to permit of expansion or contraction without fracture by providing a weak place which can afterwards be grouted up. The same principle of localising the weak places has been adopted in the construction of some retaining walls, vertical straight joints in the concrete mass being formed at regular intervals. Some persons assert that cracking is due to variation of temperature, or to expansion and contraction due to changes of temperature, unless provision is made to prevent it; but this is "not proven."

The Cracking of Concrete Walls.

If the concrete be made of cement or lime of varying quality, as may readily happen in different deliveries, the unequal character will cause a tendency to parting at the junction owing to their unequal rate of setting and unequal expansion when set. The same thing happens if the cement, although uniform in composition, is not uniformly air-slaked before use, the fresher cement expanding more than that which has been properly prepared by being spread out on a wood floor protected from the weather and turned over three or four times. The expansion varies from 0 to ¼ in. per ft., according to the amount of air-slaking it undergoes. Cracking may occur through original faults in mixing the materials to form cement. Over-liming gives high tensile strength on seven days' tests, but the free lime does not slake until the cement is partially set, and the consequent expansion is apt to disintegrate the mass. Over-clayed cement is liable to contract, and has less tensile strength. The effect of magnesia in the cement has been variously estimated. When good Portland cement has been used in concrete for sea-walls, the magnesia found has been stated to come from the sea-water, the lime being slowly washed away and the magnesia deposited in its place, at the same time causing a general expansion of the exposed part and so flaking. If the cement be overburnt, many hard particles will resist the grinding process, and only slake after much delay, when the general body of the work has set, causing blisters on the face of the work, which break off, and sometimes causing fractures in the mass. If slag be used for the aggregate, and any lime is contained in it, the same action may take place. In lime concrete, if the lime is not thoroughly slaked before using, the unslaked particles will cause local expansion when moisture reaches them, and ultimate fracture if they are in sufficient quantity.

Fracture owing to Imperfect Mixing.

Fracture may also occur from want of cohesion due to imperfect mixing of the materials, or from an occasional overdose of water in the mixing, washing away the cement. If the concrete be exposed to the sun or to drying winds as it is deposited, and the moisture be allowed to dry off too quickly, it will have a deleterious effect. If the concrete be mixed and deposited in frosty weather, or if a hard frost should set in after it is deposited, and while still "green" or imperfectly set, it is liable to cracking and partial disintegration. When concrete is thrown from a height into a trench there is a possibility of the heavier particles being separated from the others, disturbing the uniformity of the mass, and the shock also is apt to disturb the concrete below which has begun to set. It is now the practice to lower the concrete into the trenches gradually, by shoots or otherwise. If the soil or supporting surface below the concrete is not homogeneous and of equal supporting power, settlements are likely to occur which will produce fractures in the mass.

Concrete Work under Water.

At Algiers the concrete was placed in caissons formed of planking strengthened with whole timbers and lined with tarpaulin, the bottom also being formed of tarpaulin. The wooden sides were roughly cut to the contour of the bottom, and sunk in position; the concrete was then lowered in hoppers, and the sides were removed after the concrete was set. In other cases where similar blocks were required, the concrete was placed in moulds until it had set sufficiently to resist the scour of the water, and was then removed, floated out to the site, and there either sunk random for breakwaters or placed by divers for quay walls. At the New South Breakwater, Aberdeen, a somewhat similar plan of depositing liquid concrete in

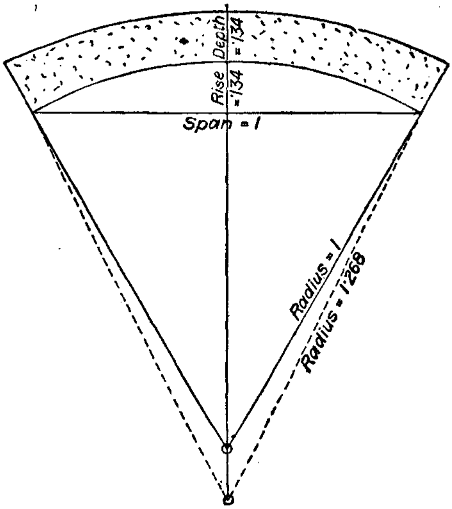


Fig. 1284.—Arch formed in Coke-Breeze Concrete.

caissons or frames was adopted. At Dublin Harbour, concrete blocks of 1 Portland cement and 7 harbour ballast were deposited by means of shears. Another method is to lower the concrete into an enclosure by means of hoppers or side shoot skips, the door being opened by a rope or chain from the surface when it has reached the required spot under water. Or the concrete may be enclosed in bags and deposited before it has set, the bags being bonded as if they were masonry blocks. Liquid concrete may also be passed down a wooden or iron pipe, a continuous flow being kept up. The "Tremie" is also used for this purpose. It is usually a pipe about 18 in. square, of wood, with a hopper top which is kept above water. This is traversed over the site by means of a travelling crane, and is kept supplied with concrete, being raised or lowered as required. Generally it is closed by a door at the bottom, and the concrete only allowed to fall through at the rate the supply can be kept up; it is thus prevented from having the cement washed out. When the concrete can set without contact with the water, there may be as much as 7 parts aggregate to 1 of cement but when it has to pass through water there should not be more than 5 to 1. The aggregate should be of mixed material so that the smaller stuff may fill the voids in the larger—say Portland cement 1 part by measure, sand and gravel 2 parts, broken stone 3 to 5 parts. A concrete mixing machine should be used, but, failing this, the materials should be measured and mixed by hand on a wooden platform; the large aggregate put down first, then the sand, then the cement, and the whole turned well over by three or four men. Then it should be watered by a rose on the end of a flexible hose, and turned over again to secure a complete mixing and thorough coating of every pebble with cement. About ½ yd. at a time is best to ensure this.

Combined Iron and Concrete.

The various systems of combining iron and concrete are now being used for many different purposes, such as the construction of piles, chimney shafts, sea-walls, groynes, tanks, etc., involving many departures from the previous methods of calculation and external design. Concrete and iron chimney shafts are at present decidedly ugly, but when designers get more used to the material they will doubtless produce better results. An example of concrete and steel chimney construction, 108 ft. high, is shown in Fig. 1285.

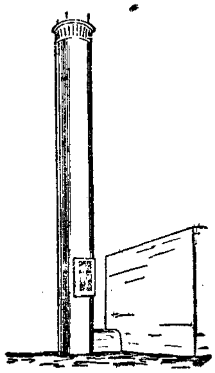


Fig. 1285.—Shaft in Concrete and Steel.

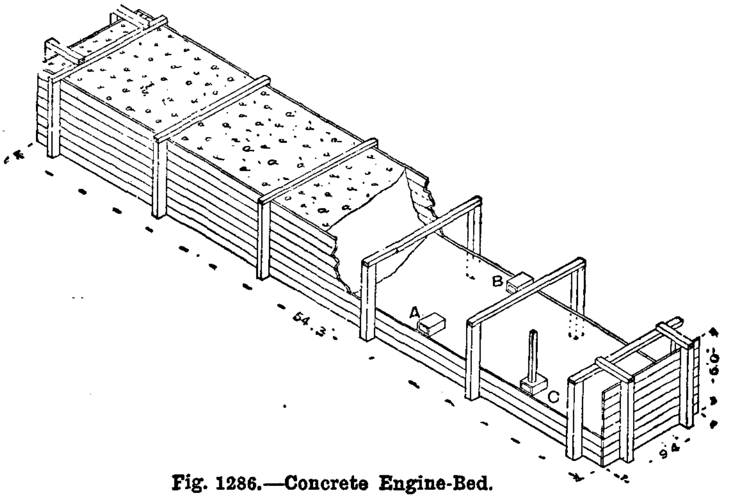


Fig. 1286.—Concrete Engine-Bed.

Concrete Bed for an Engine.

The timbers required for forming a concrete bed for an engine may be fixed by providing say 6-in. by 4-in. uprights 10 ft. apart, let 1 ft. into the ground, with cross pieces at the top to hold them to the gauge, as shown in Fig. 1286, and held also by longitudes on the top, as partly shown; then provide 9-in. by 1½-in.

boards inside the uprights to hold the concrete, and put the boards in as the work proceeds. (The size of the bed is 54 ft. 3 in. by 9 ft. 4 in. by 6 ft. thick.) When the level of the under side of the hand holes is reached, wooden boxes ABC of the required size are laid in, with upright boxes, say 3 in. by 3 in., to form the core for the bolt holes, one of which is shown in the illustration. These core boxes should be carefully fixed to the template or measurements. The concrete should be mixed to the proportion described in the specification, and put in in layers not exceeding 12 in. deep, no layer to extend more than 3 ft. in advance of the layer above. The following advice is offered in the case of a concrete pier which is to carry the part of an engine where the pulley shaft works. The engine ought not to be allowed to run until at least fourteen days after the pier is finished, but if the owner insists on using it seven days after, the only alternative is to increase the strength of the material by adding more cement, making the concrete of say 1 cement, 1 sand, and 3 hard broken brick to pass a 2-in. gauge. The cement ought to be air-slaked if the slaking has not been done before delivery ; the cement is turned out of the casks on to a wooden floor under cover and spread out, then turned over twice at intervals of say three days, if the cement is lying more than 1 ft. thick. For a concrete pier 4 ft. by 4 ft. by 7 ft. = 112 cub. ft., about 4 cub. yd. of broken brick, 1½ cub. yd. of clean sharp sand, 28 bushels of light quick-setting Portland cement, and 150 gal. of water will be required. The materials should be mixed dry in quantities not exceeding ½ yd. at a time, then turned over again while being watered through a rose, and again before loading into barrows. The concrete should be just wet enough for no water to run away, and should not be disturbed after depositing in position, but a gentle ramming done immediately will help to consolidate it.

STAIRCASES AND IRON AND STONE STEPS.

Some Definitions.

Staircases proper are made of wood; stone steps or stairs are not staircases, although often called so.

A tread is the flat part of a step.

A riser is the vertical part of one step of a staircase connecting the adjacent treads (see Fig. 1287).

The going of stairs is the horizontal distance from face to face of two consecutive risers; and sometimes the term is used for the horizontal distance occupied by the whole flight; that is, the horizontal distance between the first and last risers.

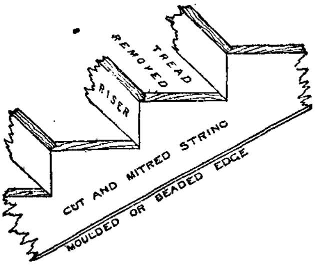


Fig. 1287.—Riser, Cut and Mitred String, etc.

A ramp (in handrailing) is a vertical bend in a handrail, concave on top, as where it leaves the straight for a rise.

A knee is a vertical bend, convex on top, as where it leaves a rising direction for the horizontal.

A swan-neck is the conjunction of a ramp and knee.

A wreath is a horizontal bend in a handrail, as where it turns round the corner of a hanging passage.

A twist or wrythe is a spiral curve in a handrail where made to suit winders on a circular-ended well-hole; but the term wreath is commonly used for the latter case.

Proportions of Treads to Risers.

Subtract twice the rise from 23 in., and the result will be the proper tread; or take the tread from 23 in., and half the remainder will be the rise. By the graphic method, draw a right-angled triangle 12 in. high and 24 in. long, as Fig. 1288, then any height of riser (x) measured on the vertical line will give the proper width of tread by horizontal measurement (y) to the inclined line. Other rules are: Tread + rise = 16 to 17; tread × rise = 60 to 65; tread + twice rise = 22 to 25.

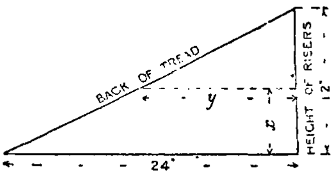


Fig. 1288.—Diagram showing Proportions of Tread and Riser.

$$\frac{\text{Angle of string in degrees} + 42}{12} = \text{rise}.$$

$$\frac{96 - \text{angle of string}}{6} = \text{tread}.$$

Types of Staircases.

Straight staircases (Fig. 1289) have all the steps parallel to one another and rising in the same direction, with or without an intermediate landing.

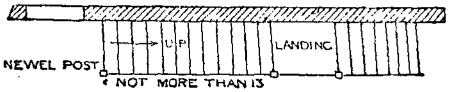


Fig. 1289.—Plan of Straight Staircase.

Dog-legged staircases are always divided into two portions having the same or different lengths, but generally with the first portion containing the greater number of steps to allow of headway below, and the second part always leading in the opposite direction. The inner ends of the steps in each flight are plumb with those in the other. The junction is made by a landing, or a quarter-space and winders, or winders only, as in Fig. 1290. The framing of a dog-legged staircase is shown in Fig. 1291.

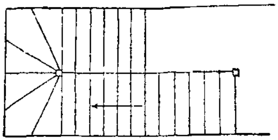


Fig. 1290.—Plan of Dog-legged Staircase.

A dog-legged staircase 3 ft. 6 in. wide, 12 ft. high from floor to floor, and having landing 7 ft. above lower floor, is shown in plan by Fig. 1292, and in corresponding section by Fig. 1293. Fig. 1294 shows detail of one step and fixing to wall string. An alternative method of attaching the short joists to the pitching piece would be to use a tusk tenon joint.

Open newel staircases are very similar to the dog-legged, but at the junction of the flights two newels occur, allowing a space between the plumb of the inner edges of the stairs, as in Fig. 1295.

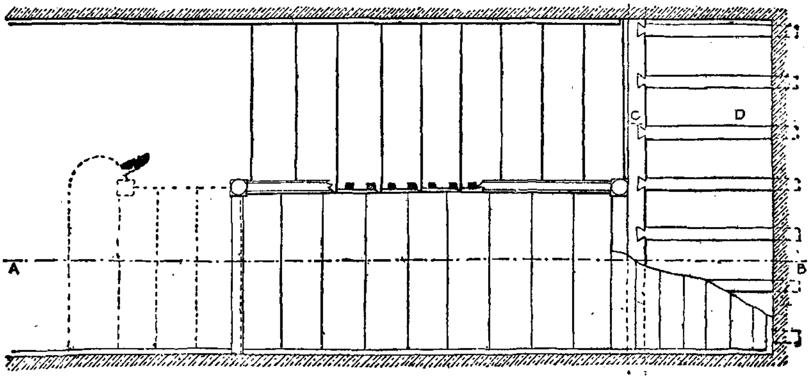


Fig. 1292.—Plan of Dog-legged Staircase 3 Ft. 6 In. wide.

Geometrical staircases (Fig. 1296) are similar to the last in plan, but continuous from bottom to top, without newels. The flights are arranged round a well-hole, and generally with a curtail step at bottom. The framing of a geometrical staircase is shown in Fig. 1297.

Circular-newel staircases are usually cast-iron steps arranged round a central wrought-iron newel, as in Fig. 1298; or in stone they are contained within a circular wall, as of a turret, and their inner ends superposed form a newel, as in Fig. 1299

Circular geometrical staircases are somewhat similar, but occur usually in wood or iron, and have a circular well in place of the newel post, as in Fig. 1300.

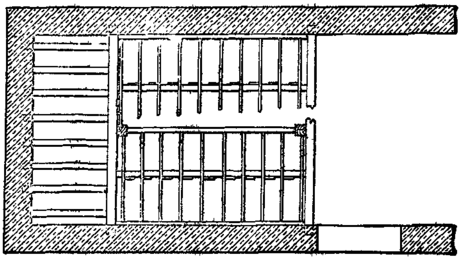


Fig. 1291.—Framing of Dog-legged Staircase.

A Bracketed Staircase has ornamental brackets on the outer string under the projecting nosings of the treads, as shown in the view of part of a staircase given by Fig. 1301.

Curtail Step.

A curtail step is the bottom step in a flight, which is made to project from the line of the upper steps to carry a newel post with a scroll handrail, as in Fig. 1302. The method of setting out a curtail step is made clear in Fig. 1303.

Cornice Moulding under Flewing Soffit.

A cornice is not usually run under winding stairs, owing to the impossibility of making the

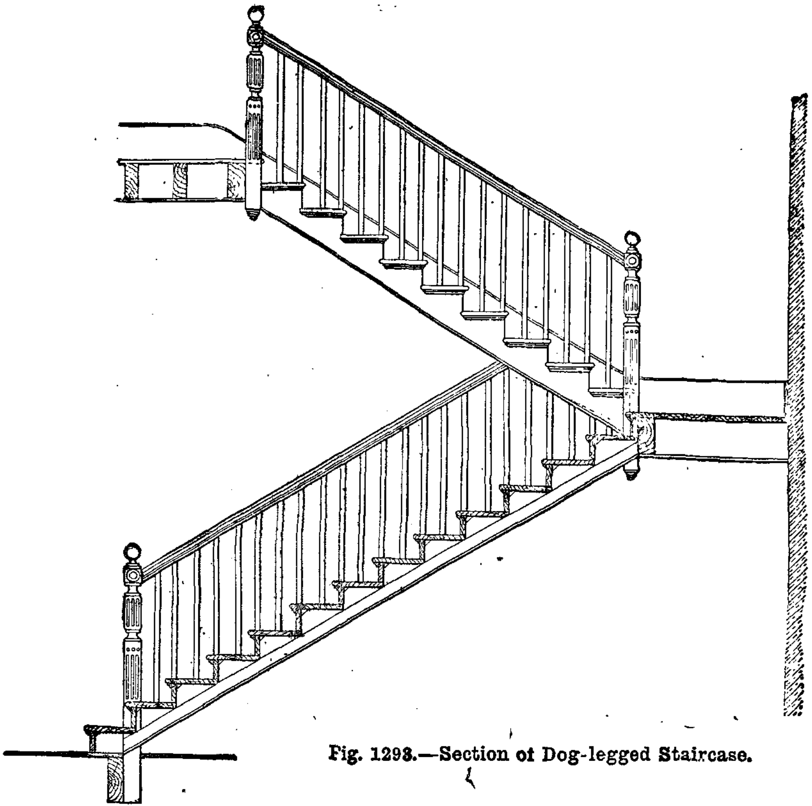


Fig. 1293.—Section of Dog-legged Staircase.

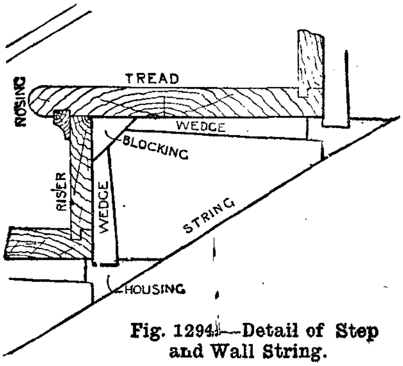

NOSING
TREAD
WEDGE
BLOCKING
RISER
WEDGE
STRING
HOUSING

Fig. 1294.—Detail of Step and Wall String.

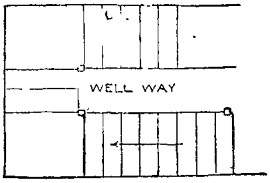


Fig. 1295.—Plan of Open Newel Staircase.

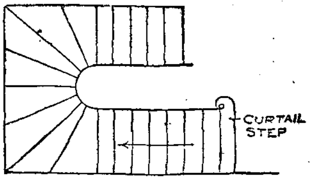

CURTAIL STEP

Fig. 1296.—Plan of Geometrical Staircase.

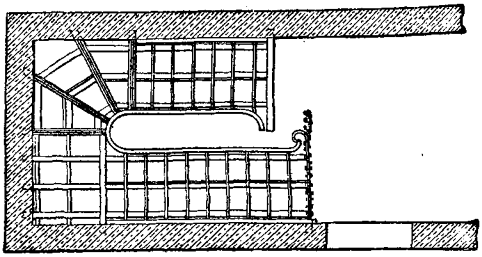
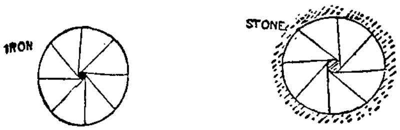


Fig. 1297.—Framing of Geometrical Staircase.



Figs. 1298 and 1299.—Plans of Circular-Newel Staircase.

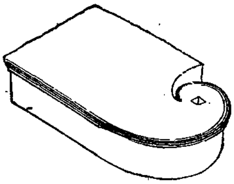


Fig. 1302.—Curtail Step.

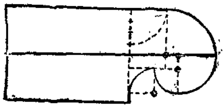


Fig. 1303.—Setting-out Curtail Step.

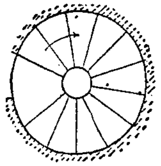


Fig. 1300.—Plan of Circular Geometrical Staircase.

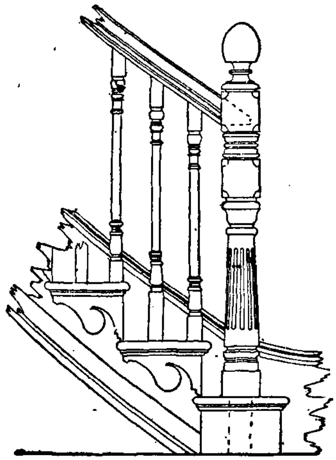


Fig. 1301.—Bracketed Staircase.

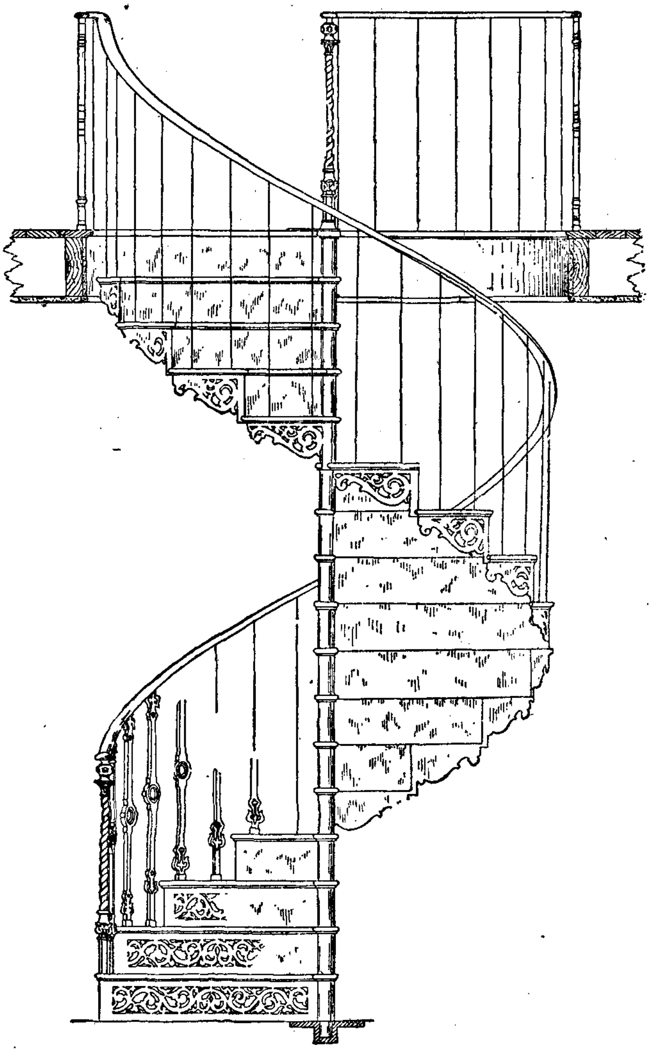


Fig. 1306.—Elevation of Iron Spiral Staircase.

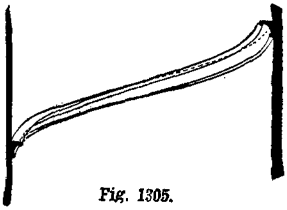


Fig. 1305.

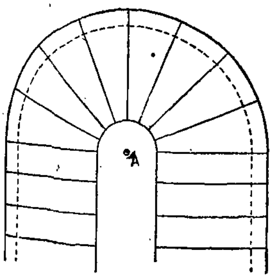


Fig. 1304.

Fig. 1304.—Plan of Geometrical Staircase. Fig. 1305.—Elevation of Cornice under Soffit.

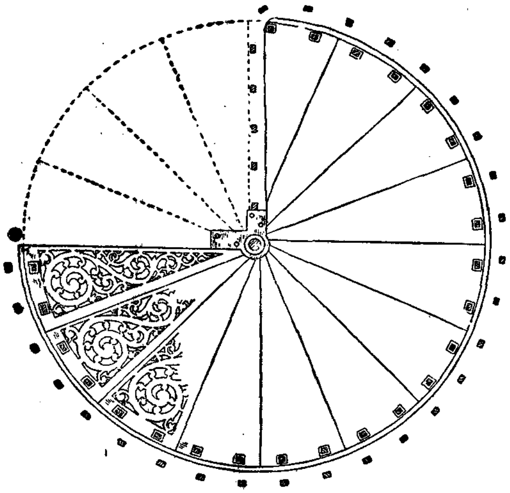
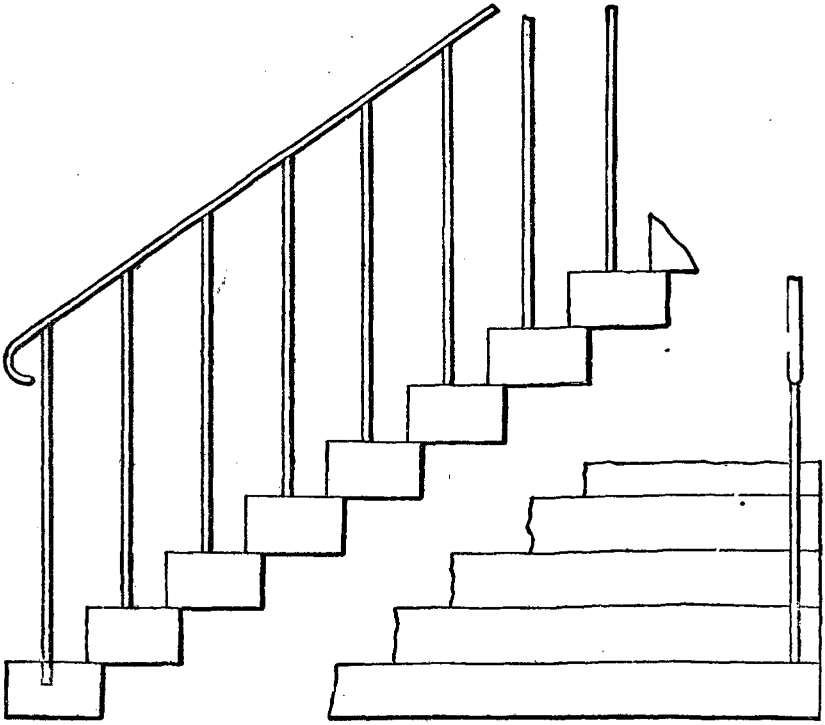


Fig. 1307.—Plan of Iron Spiral Staircase.

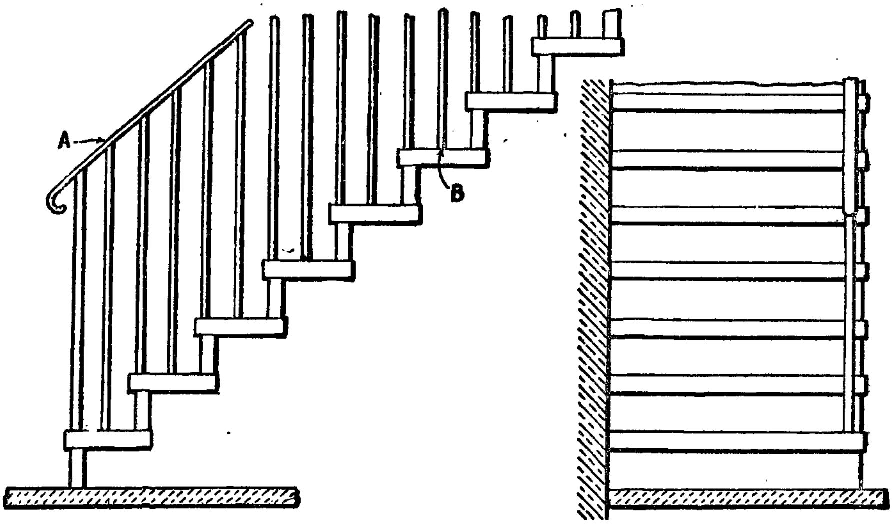
same section fit, unless the stairs are in a circular angle, when the mould can be attached to a radius rod having an iron ferrule or a short piece of gas-pipe on the end, sliding up and down a smaller diameter gas barrel, fixed vertically at the centre from which the curve is struck. Figs. 1304 and 1305 show what is meant. The object is to keep the mould vertical while sweeping it along a spiral surface. Fig. 1304 shows a geometrical staircase with the centre at A from which the radius rod would work, and Fig. 1305 an elevation of a simple cornice, projected on the usual system of a spiral line.

Iron Spiral Staircase.

Fig. 1306 shows an iron spiral staircase having seventeen steps; Fig. 1306 represents the elevation, and Fig. 1307 the plan. The castings for the steps are threaded on a wrought-iron central rod. An iron spiral staircase, 6 ft. in diameter, rising 14 ft., would have twenty steps rising 8·4 in. each, or twenty-one steps rising 8 in. each, and the tread would be 13 in. wide in the centre. Modifications may be made in the design to suit any position of landing at top and bottom.



Figs. 1308 and 1309.—Elevations of Stone Warehouse Steps.



Figs. 1310 and 1311.—Elevations of Stone Basement Steps.

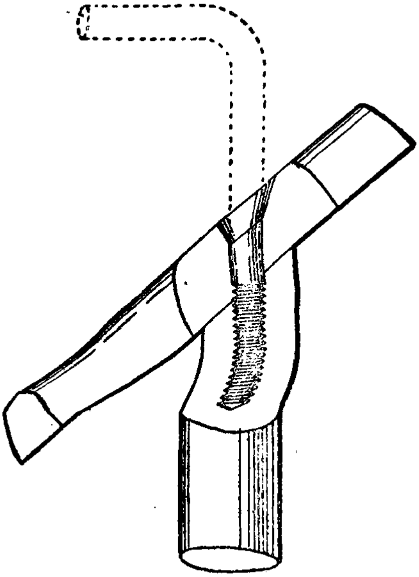


Fig. 1312.—Fixing Baluster to Hand Rail.

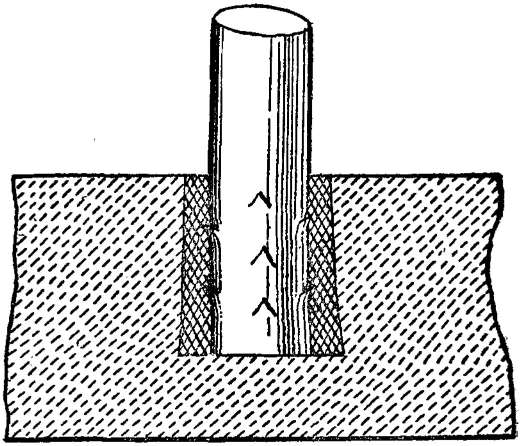


Fig. 1313.—Fixing Baluster to Stone Step.

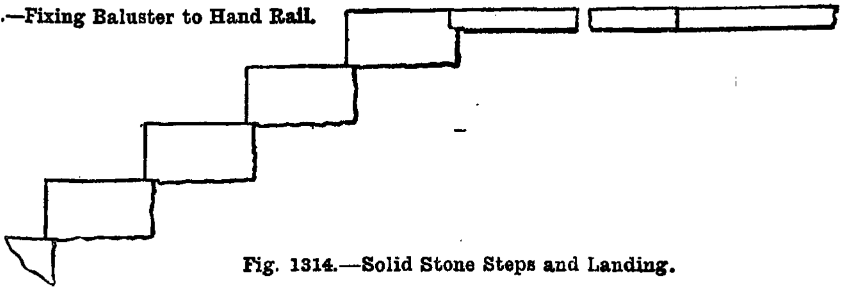


Fig. 1314.—Solid Stone Steps and Landing.

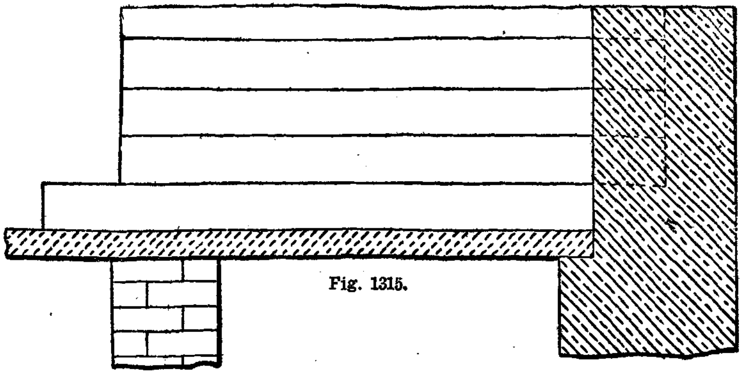


Fig. 1315.

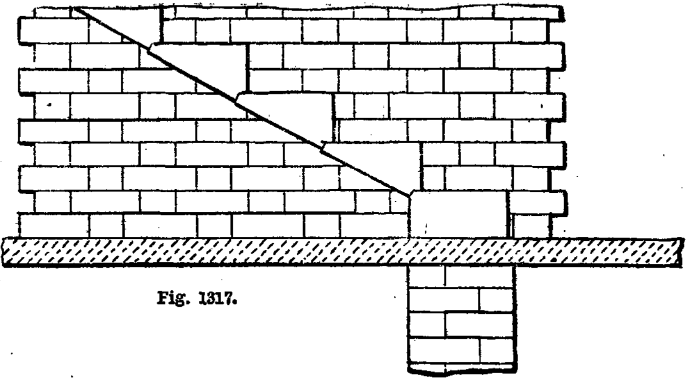


Fig. 1317.

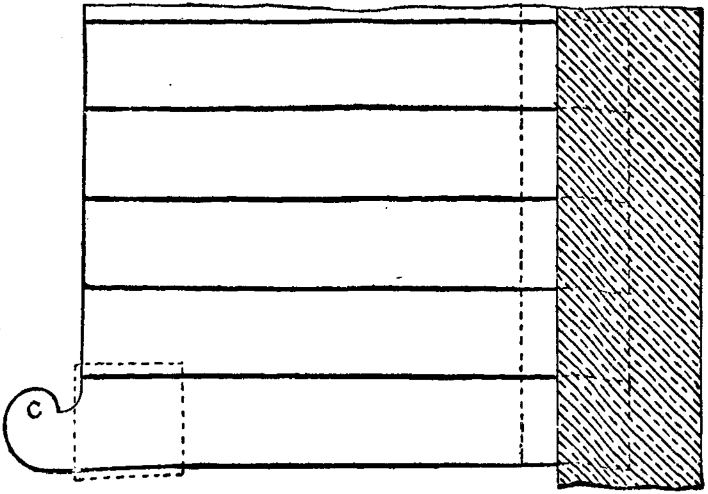


Fig. 1316.

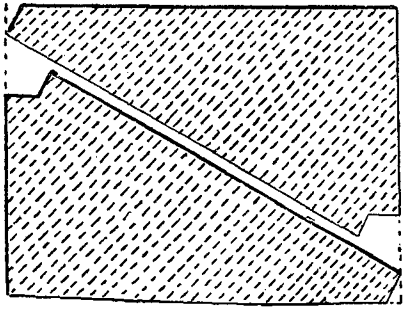


Fig. 1318.

Figs. 1315 to 1317.—Front Elevation, Plan and Side Elevation of Feather-edged Steps. Fig. 1318.—Section showing how Two Steps are Cut from One Block.

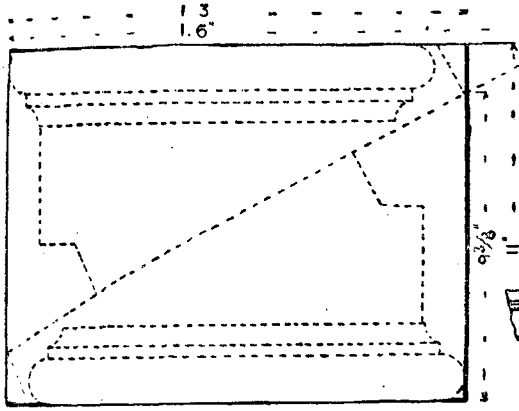


Fig. 1320.—Cutting Moulded Feather-edged Steps.

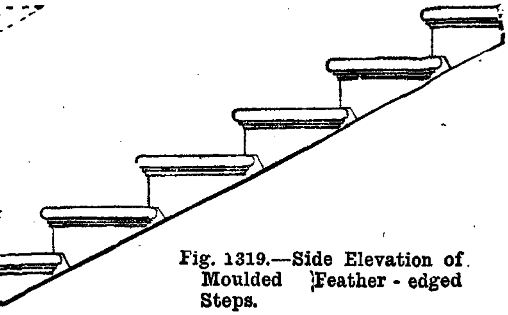
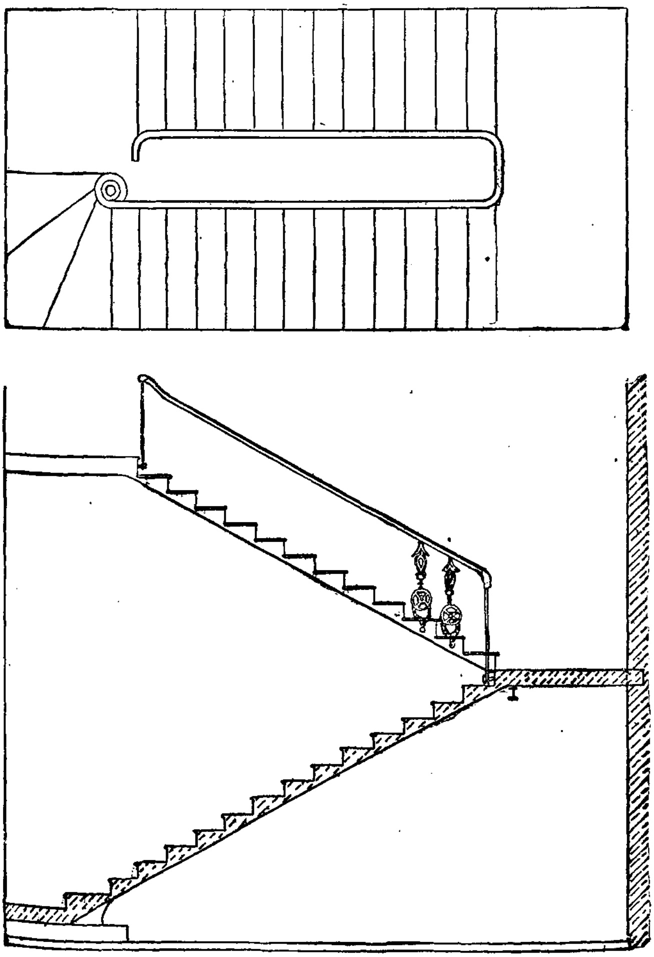


Fig. 1319.—Side Elevation of Moulded Feather-edged Steps.



Figs. 1321 and 1322.—Plan and Section of Stone Staircase.

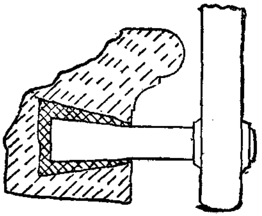


Fig. 1323.—Baluster fixed to Stone Step.

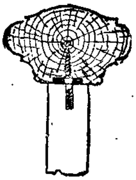


Fig. 1324.—Handrail fixed to Baluster.

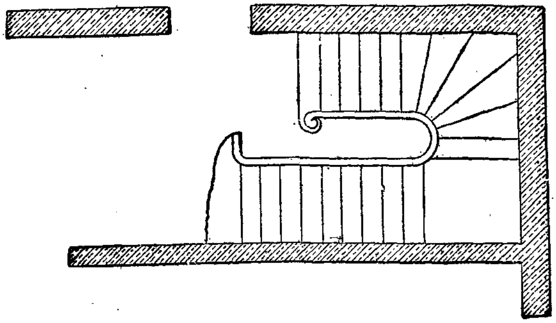


Fig. 1325.—Plan of Geometrical Stair with Quarter Space.

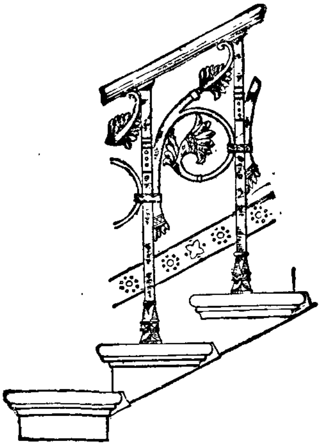


Fig. 1327.—Part Side Elevation of Overhanging Stone Stair.

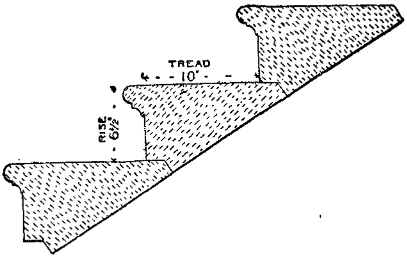


Fig. 1326.—Section through Stone Steps.

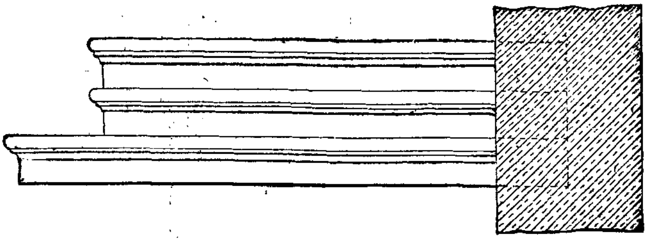


Fig. 1328.—Elevation of Overhanging Stone Steps.

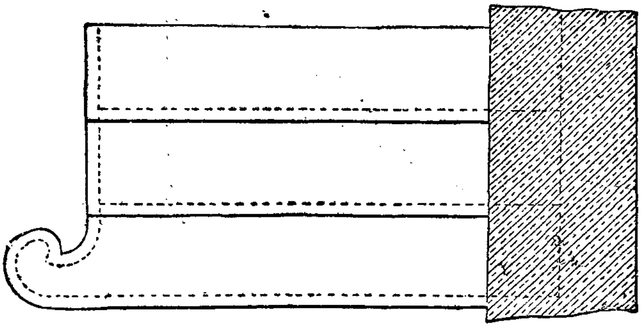
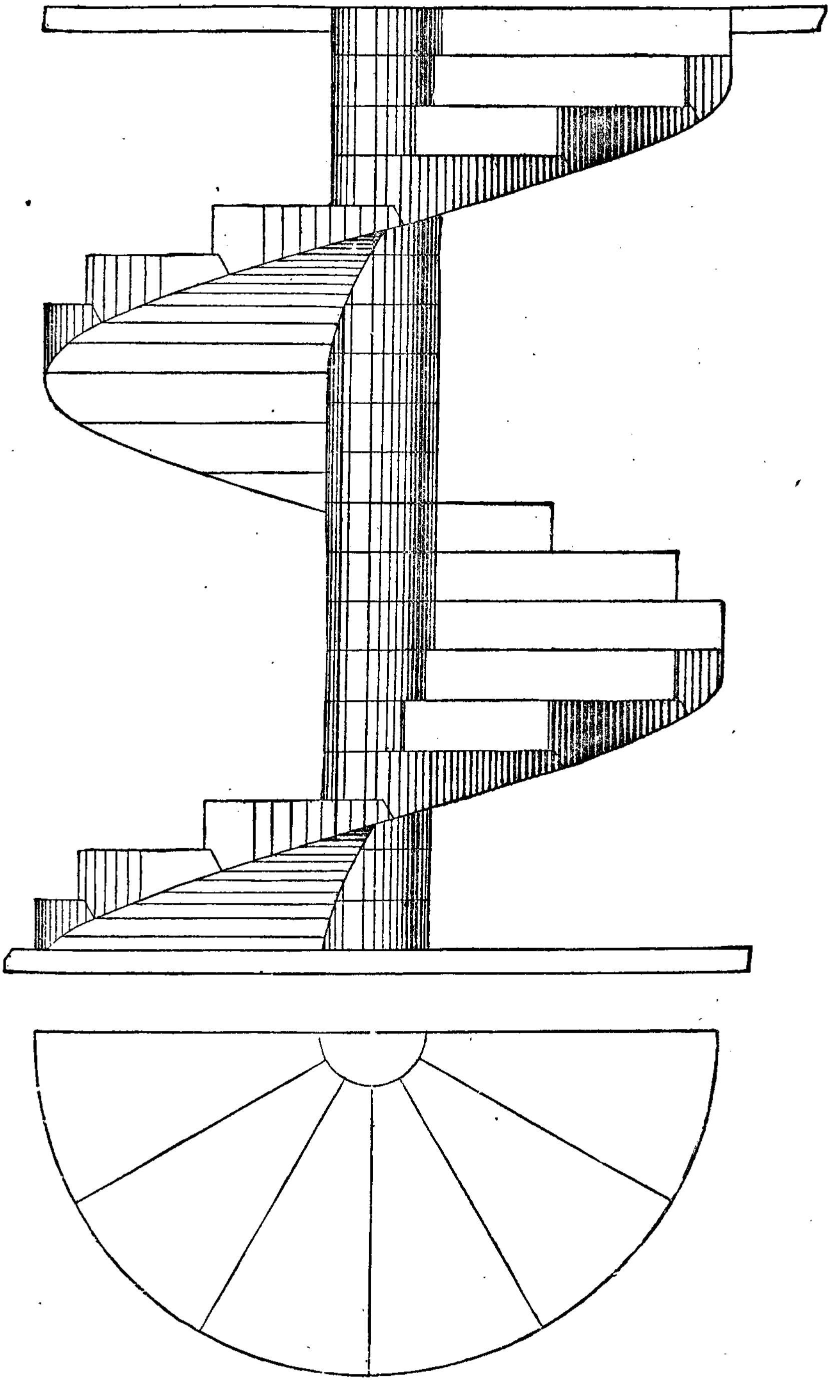


Fig. 1329.—Plan of Overhanging Stone Steps.



Figs. 1330 and 1331.—Elevation and Half-plan of Spiral Stone Staircase.

Simple Stone Steps.

Warehouse steps are shown in side and front elevation by Figs. 1308 and 1309, the basement steps of a house by Figs. 1310 and 1311, details of the fixings at A and B (Fig. 1310) being given by Figs. 1312 and 1313. Fig. 1314 shows solid stone steps and landing. Front elevation, plan and side elevation of feather-edged steps are presented by Figs. 1315 to 1317; a sectional view showing how two steps are cut from a block is given by Fig. 1318. Moulded feather-edged steps are shown in Fig. 1319; they are cut out in pairs, as shown in Fig. 1320.

Stone Staircases.

Fig. 1321 is a plan and Fig. 1322 a sectional elevation of stone steps placed in a stairway 20 ft. by 10 ft., the height from floor to top of landing being 15 ft. The steps are 4 ft. long. The way in which the balusters are fastened to the stone steps is shown in Fig. 1323. The handrail may be of the section illustrated in Fig. 1324, which shows the method of fastening the handrail to the balusters. A geometrical stair with a quarter space is shown by Fig. 1325; this stair is designed for a height of 11 ft. 6 in. from floor to floor; a section through three of the steps is presented by Fig. 1326. Part side elevation of an overhanging stone stair, 4 ft. wide, suitable for first-class work, is shown by Fig. 1327; the steps are feather-edged or spandril and are fully moulded, the balusters being of an ornamental character. Fig. 1328 is a front elevation, and Fig. 1329 is a plan to correspond. Elevation and half-plan of a spiral stone staircase are presented by Figs. 1330 and 1331.

Rolled Joist Stringer for Stairs.

In determining the size of a rolled steel joist acting as a stringer to a fireproof staircase, the rolled joists must be considered as supported at both ends and carrying a distributed load equal to half the weight of the stairs and the landings + 1 cwt. per ft. super. of tread area for external load. Half this total load will be the reaction at each end, and the joint at the bend must have double covers securely bolted on each side of the web and fitting closely between the flanges. The depth of the joist should be not less than one-twenty-fourth of the full length.

BRIDGES.

Timber Footbridges.

THE distance which a simple 9-in. by 3-in. plank will safely span in the form of a footbridge is limited to about 12 ft. For any span beyond this a trussed beam must be used. In the case of a footbridge 64 ft. long, 2 ft. wide, supported at the centre and at 2 ft. from each end, the simplest method is to truss the bridge above and arrange it in the manner shown by Figs. 1332 and 1333 (the detail at A is shown enlarged in Fig. 1334). The same scantlings and general construction answer for a portable gangway having a total length not exceeding 35 ft. A cattle bridge for a span of 41 ft. over a stream is shown in Figs. 1335 to 1337. This requires substantial brickwork or cement concrete abutments, carried down to a good bottom where they would be free from scour. In timber bridges allowance has always to be made for a certain amount of neglect and unavoidable decay, so that a fair margin of safety must be allowed to commence with. Creosoted timber would be most suitable; but

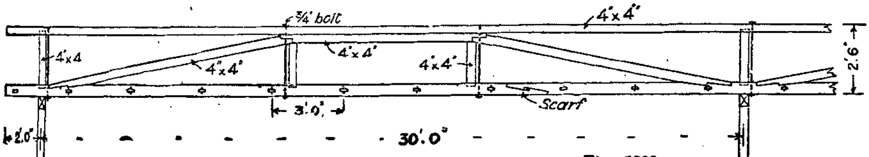

¾" bolt
4"x 4"
4"x4
4"x4"
4"x 4"
4"x4"
2'.6"
3'.0"
Scarf
1'.0"
30'.0"

Fig. 1332.

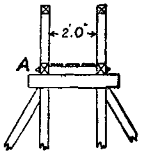


Fig. 1333.

Figs. 1332 and 1333.—Simple Timber Footbridge.

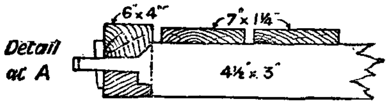

Detail at A
6"x 4"
7"x 1¼"
4½"x 3"

Fig. 1334.

Fig. 1334.—Tusk-tenon Joint in Footbridge.

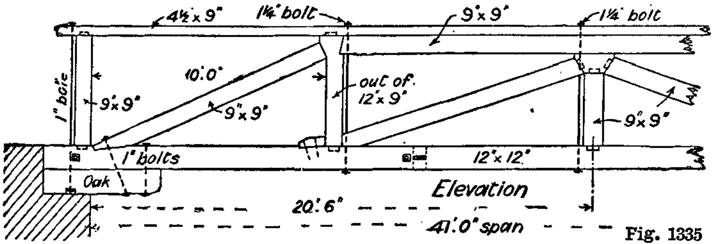

4½"x 9"
1¼" bolt
9"x 9"
1¼" bolt
1" bolt
9"x 9"
10'.0"
9"x 9"
out of 12"x 9"
9"x 9"
1" bolts
12"x 12"
Oak
Elevation
20'.6"
41'.0" span

Fig. 1335

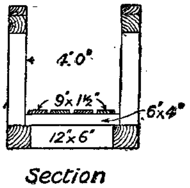

4'.0"
9"x 1½"
6"x4"
12"x 6"
Section

Fig. 1336.

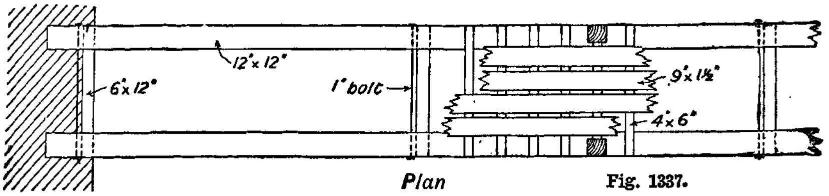

12"x 12"
6"x 12"
1" bolt
9"x 1½"
4"x 6"
Plan

Fig. 1337.

Figs. 1335 to 1337.—Elevation, Section, and Plan of Cattle Bridge.

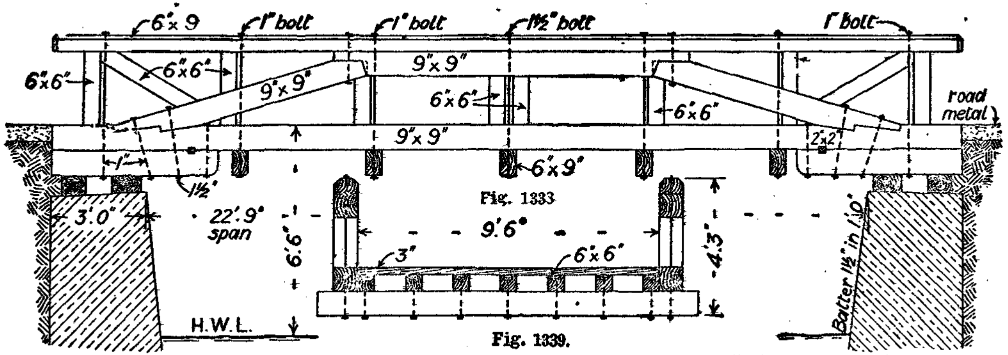

6"x 9
1" bolt
1" bolt
1½" bolt
1" bolt
6"x6"
6"x6"
9"x 9"
9"x9"
6"x6"
6"x6"
road metal
9"x 9"
2"x2"
6"x9"
1"
1½"
3'.0"
22'.9" span
6'.6"
H.W.L.
Fig. 1333
9'.6"
3"
6"x6"
4'.3"
Batter 1½" in 1'.0"

Fig. 1339.

Figs. 1338 and 1339.—Elevation and Cross Section of Footbridge.

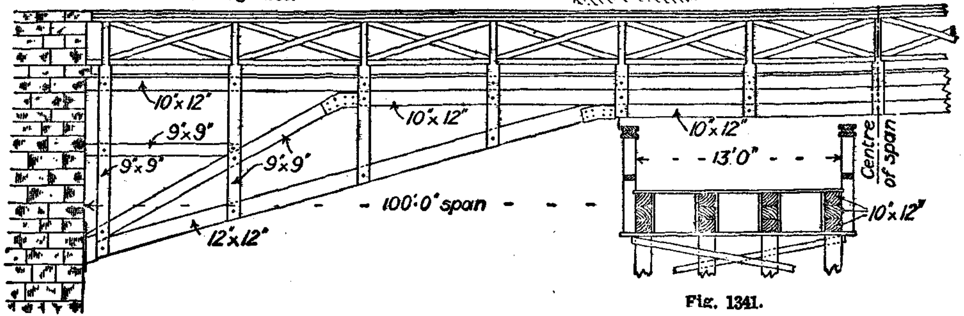
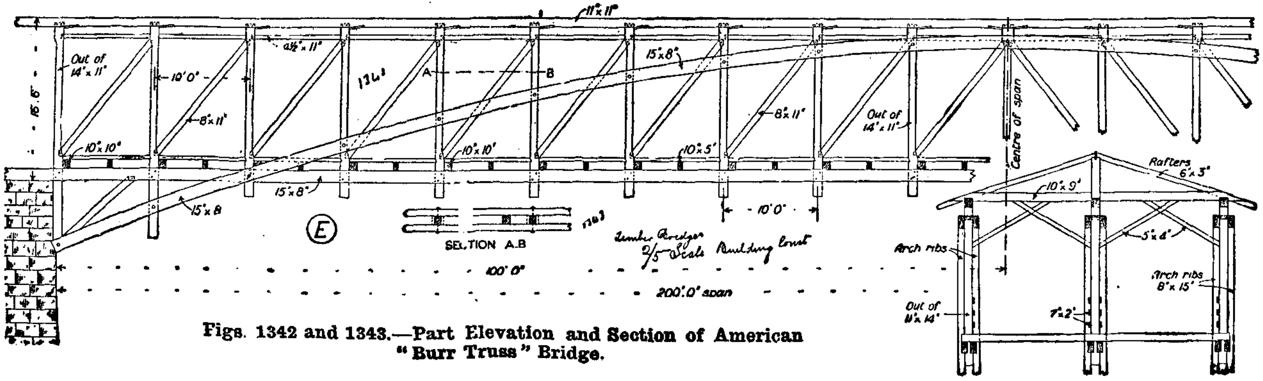

10"x 12"
9"x9"
9"x9"
9"x9"
10"x 12"
10"x 12"
12"x12"
100'.0" span
13'.0"
Centre of span
10"x 12"

Fig. 1341.

Figs. 1340 and 1341.—Part Elevation and Section of Telford Bridge.


11"x 11"
Out of 14"x 11"
10'.0"
4½"x 11"
1341
A
B
15"x8"
8"x11"
8"x 11"
Out of 14"x 11"
Centre of span
16'.6"
10"x 10"
10"x 10"
10"x 5"
15"x8"
15"x 8
E
SECTION A.B
1343
10'.0"
Timber Bridges 2/5 Scale Building Const
100'.0"
200'.0" span
10"x 9"
Rafters 6"x 3"
5"x 4"
Arch ribs
Arch ribs 8"x 15"
7"x2"
Out of 11"x 14"

Figs. 1342 and 1343.—Part Elevation and Section of American "Burr Truss" Bridge.

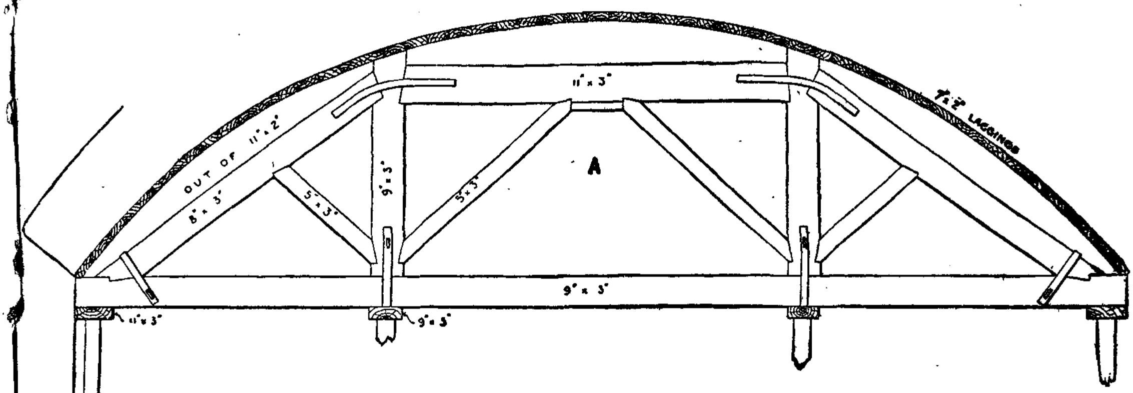
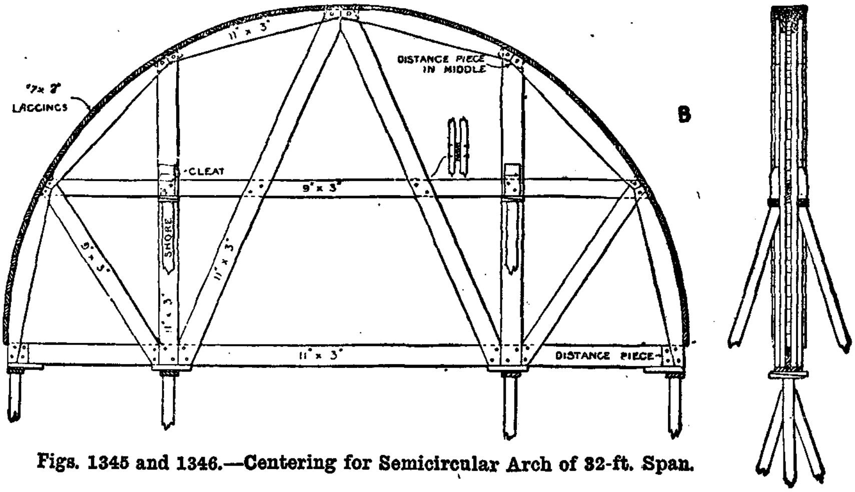

11"×3"
7"×2" LAGGINGS
OUT OF 11"×2"
8"×3"
5×3"
9"×3"
5×3"
A
9"×3"
11"×3"
9"×3"
5"×5"

Fig. 1344.—Centering for Railway Viaduct Arch of 25-ft. Span.


11"×3"
DISTANCE PIECE IN MIDDLE
7×2" LAGGINGS
B
CLEAT
9"×3"
SHORE
9"×3"
11"×3"
11"×3"
11"×3"
DISTANCE PIECE

Figs. 1345 and 1346.—Centering for Semicircular Arch of 32-ft. Span.

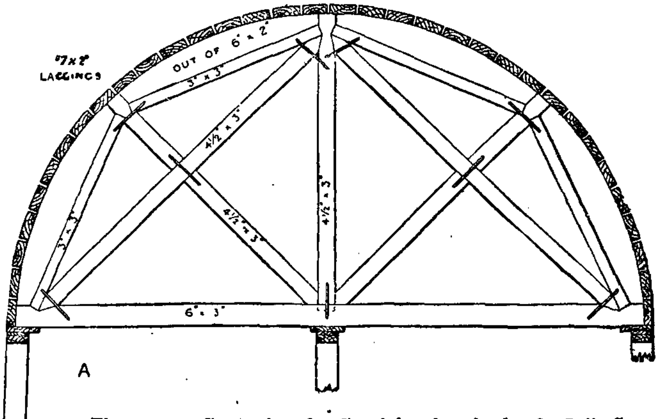

7×2" LAGGINGS
OUT OF 6"×2"
3"×3"
4½×3"
4½×3"
4½×3"
3"×3"
6"×3"
A

Fig. 1347.—Centering for Semicircular Arch of 15-ft. Span.

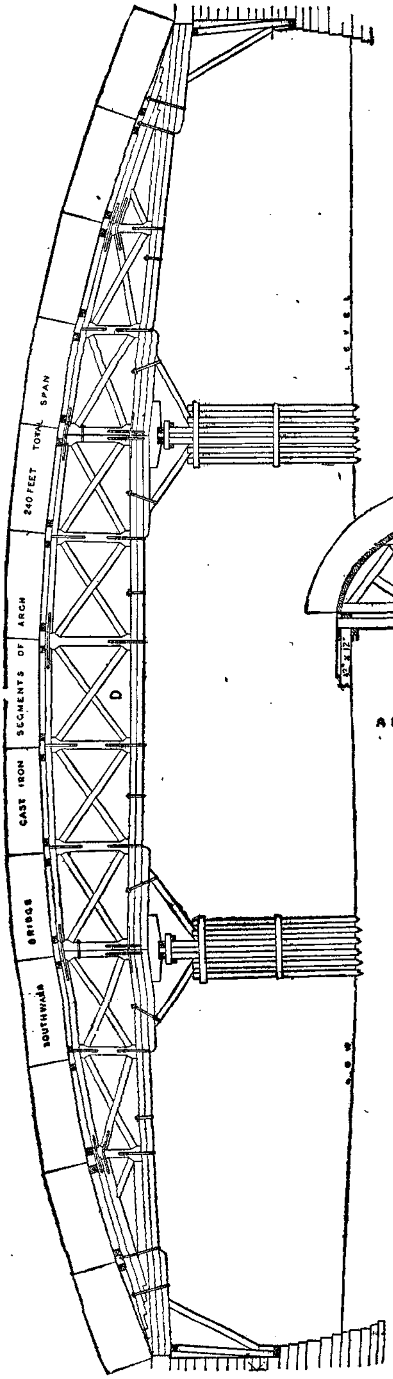

SOUTHWARK BRIDGE CAST IRON SEGMENTS OF ARCH 240 FEET TOTAL SPAN
D
LEVEL

Fig. 1351.—Centering for Spans of Southwark Bridge.

failing that, the wood may be painted with two coats of Stockholm tar. Another footbridge—somewhat similar to the last—is shown by Fig. 1338, in which the scantlings indicated would be suitable for a span of nearly 23 ft. A cross section of the bridge is presented by Fig. 1339.

Wooden Bridges for 100-ft. and 200-ft. Span.

The Telford bridge shown in half elevation by Fig. 1340 and in cross section by Fig. 1341 is suitable for a span of 100 ft. if the scantlings indicated are adopted. An American "Burr Truss" wooden bridge is illustrated by Fig. 1342, and a cross section being presented by Fig. 1343. A section on the line A B through two posts is given in Fig. 1342. This bridge, with the scantlings shown, is suitable for as great a span as 200 ft.

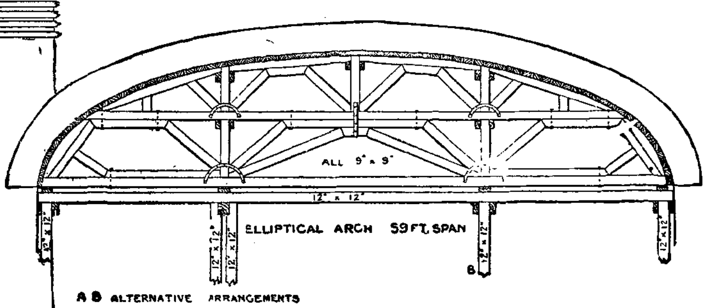
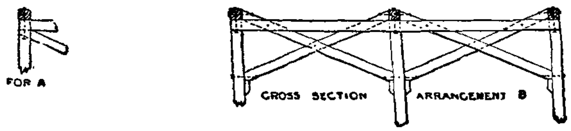

ALL 9" x 9"
12" x 12"
ELLIPTICAL ARCH 59 FT. SPAN
12" x 12"
12" x 12"
12" x 12"
12" x 12"
B
A B ALTERNATIVE ARRANGEMENTS

Fig. 1348.—Centering for Elliptical Arch of 59-ft. Span.


FOR A
CROSS SECTION ARRANGEMENT B

Figs. 1349 and 1350.—Alternative Arrangement of Supports for Centering.

Centerings for Brick and Masonry Bridges and Large Arches.

The centering used for railway viaduct arches is shown by Fig. 1344, this being suitable for a 25-ft. span. The centering shown by Figs. 1345 and 1346 is for a semicircular arch of 32-ft. span in an 18-in. wall, and that shown by Fig. 1347 for an arch of the same shape, but of 15-ft. span. The centering for an elliptical arch of 59-ft. span is illustrated by Fig. 1348, in which alternative arrange-

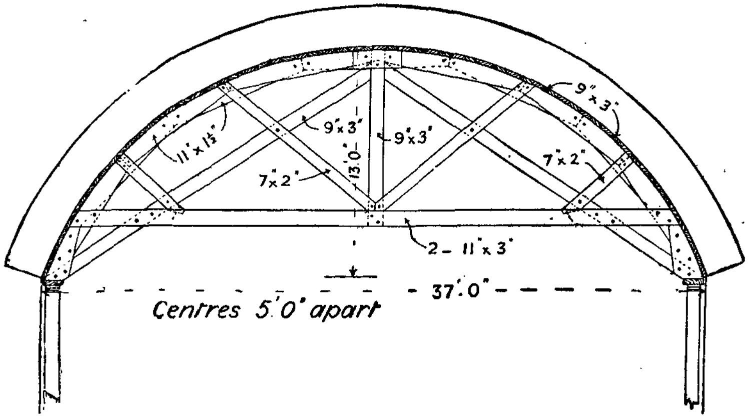

9" x 3"
11" x 1½"
9" x 3"
9" x 3"
13'.0"
7" x 2"
7" x 2"
2 - 11" x 3"
Centres 5'.0" apart
37'.0"

Fig. 1352.—Centering for Road Bridge Arch.

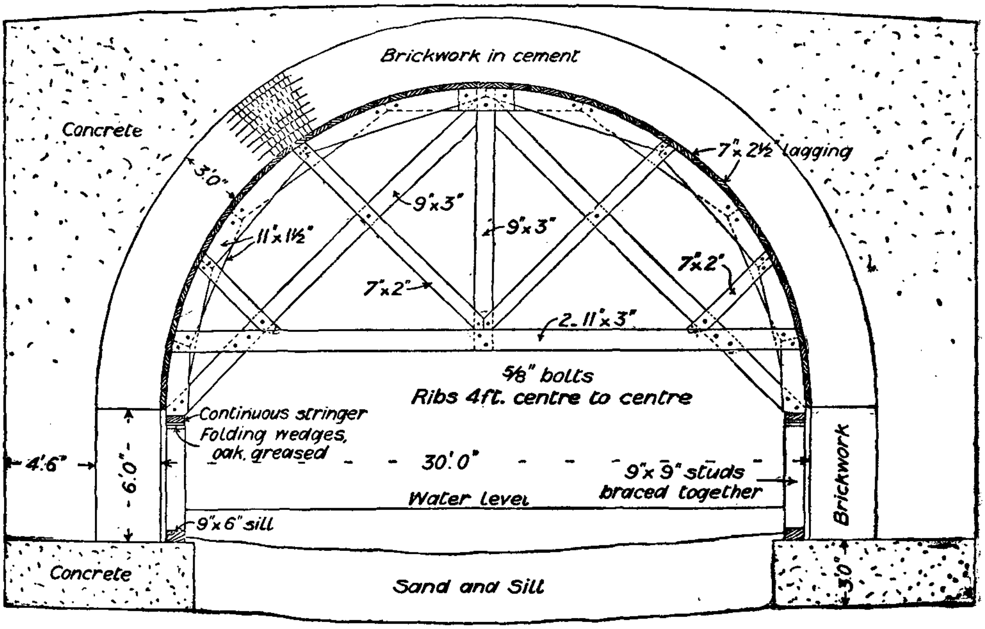
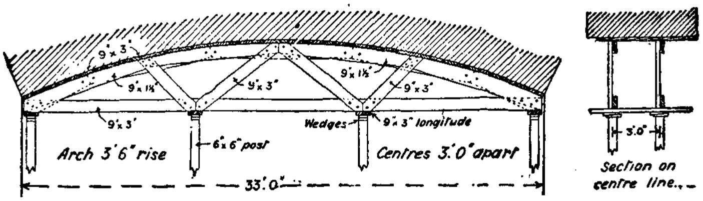

Brickwork in cement
Concrete
7" x 2½" lagging
3'.0"
9" x 3"
9" x 3"
11" x 1½"
7" x 2"
7" x 2"
2- 11" x 3"
⅝" bolts
Ribs 4ft. centre to centre
Continuous stringer
Folding wedges, oak, greased
4'.6"
6'.0"
30'.0"
9" x 9" studs braced together
Water Level
Brickwork
9" x 6" sill
Concrete
Sand and Silt
3'.0"

Fig. 1353.—Centering, Brick Arch and Concrete.


9" x 3"
9" x 1½"
9" x 3"
9" x 1½"
9" x 3"
9" x 3"
Wedges
9" x 3" longitude
6" x 6" post
Arch 3'6" rise
Centres 3'.0" apart
33'.0"
3'.0"
Section on centre line.

Figs. 1354 and 1355.—Elevation and Section of Centering for Masonry Arch, 33-ft. Span and 3-ft. 6-in. Rise.

ments of the supports are shown at A and B, these being further shown (in cross section) by Figs. 1349 and 1350 respectively. The centering shown in Fig. 1351 is of a more ambitious nature; it is adapted for a total span of 240 ft., and is the one used for the spans of Southwark Bridge, London. A suitable centering for the arch of a bridge over which a road for the usual traffic is to be laid is shown by Fig. 1352. The span is 37 ft. in the clear with a rise of 7 ft. If any deflection occurs in the centre of the span from the yielding of the timbers or fastenings, an additional row of posts and wedges may be put in. Either the bridge should be purely of brick, or the whole face of the bridge should be cased in stone. Fig. 1353 shows a similar centering supporting a brick arch, 3 ft. thick, with concrete over it. The beam is kept as high as possible consistently with safety, so as to be clear above objects floating on the river at flood time. The arch is a very substantial one, and should rest on a good foundation. The concrete would be better if deeper. Fig. 1354 illustrates the arrangement of centering for a masonry arch 33-ft. span, 3-ft. 6-in. rise. The rise is very little, and the thrust on the abutments of the arch will therefore be proportionately great. Fig. 1355 shows the section on the centre line.

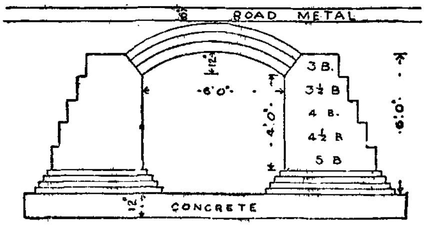

ROAD METAL
3 B.
3½ B
4 B.
4½ B
5 B
6'0"
4'0"
6'0"
CONCRETE

Fig. 1357.

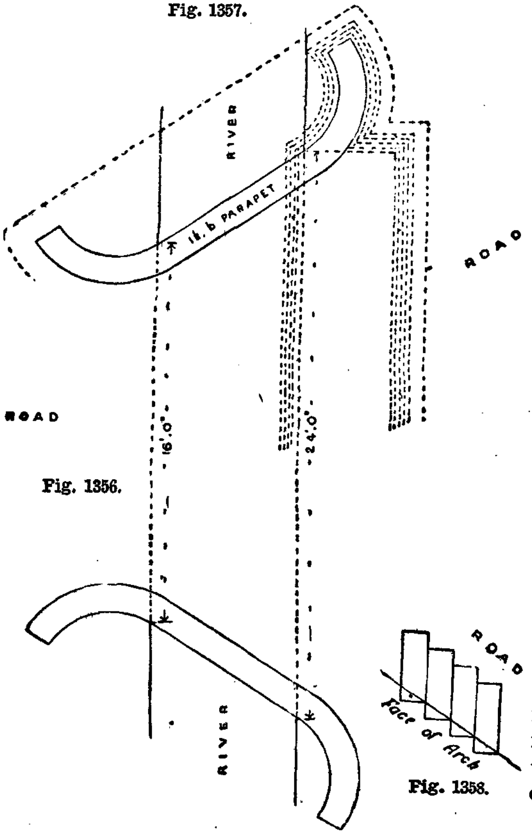

RIVER
14 B PARAPET
ROAD
ROAD
16'0"
24'0"
RIVER
ROAD
Face of Arch

Fig. 1356.

Fig. 1358.

Figs. 1356 to 1358.—Brick Arch under Roadway.

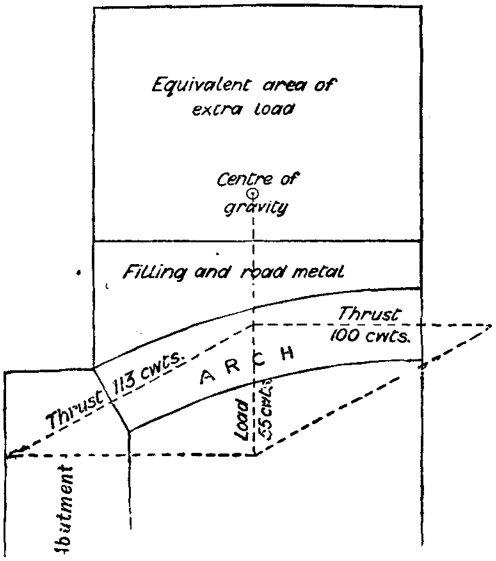

Equivalent area of extra load
Centre of gravity
Filling and road metal
Thrust 100 cwts.
Thrust 113 cwts.
ARCH
Load 55 cwts.
Abutment

Fig. 1359.—Diagram showing Brick Arched Highway Bridge.

Brick Arch under Roadway.

It is required to build a brick bridge to be carried splayed over a river to meet the two roads as shown in Fig. 1356. This is a very unusual case, and the better plan would probably be to place the junction of the two roads (on the farther side of the bridge) farther

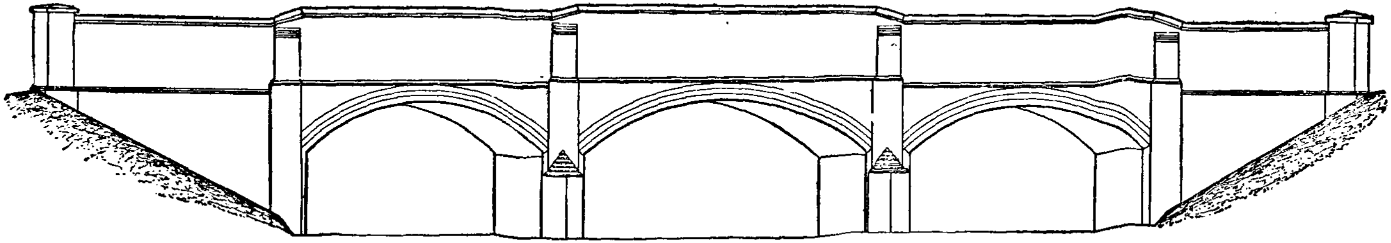


Fig. 1360.—Elevation of Brick Bridge, 86-ft. Total Span.

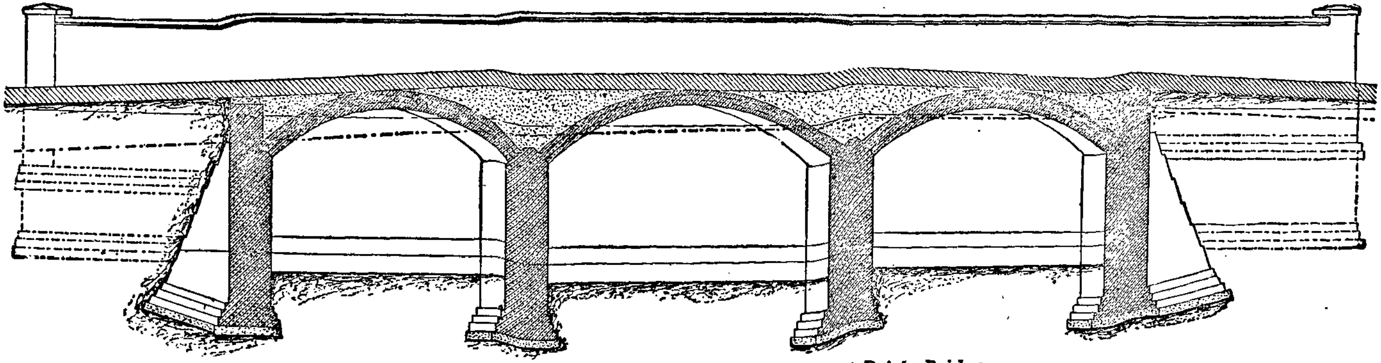


Fig. 1361.—Longitudinal Sectional Elevation of Brick Bridge.

back so that an ordinary straight bridge could be made. However, Fig. 1356 shows the plan, and Fig. 1357 the section of the proposed arrangement. The earth should be well rammed behind the abutments, and the bricks of the arch rings should be laid as shown in Fig. 1358, the projecting angles being cut off.

Brick Arched Highway Bridge.

Assume that a highway bridge is to be erected over a stream, and the bridge must be designed to bear the passage of traction engines and similar heavy traffic. First put the bridge down to scale, adding an equivalent height for the external load, say 5 cwt. per ft. super., all at 1 cwt. per cubic foot. Then find the centre of gravity of the arch and the load and draw the parallelogram of forces, taking the load vertically and the thrusts at right angles to the crown and impost. From the end of the load line draw parallels so as to complete the parallelogram, and scale off. The greatest thrust will be on the impost = 113 cwt. for 1 ft. run of arch, which for four half-brick rings will be equal to say $3\frac{3}{4}$ tons per square foot, and is not too much for good brickwork. The abutment must be strong enough to resist the thrust, and should be well backed up by ramming the earth in layers. Fig. 1359 will afford a clearer understanding of what is meant by the above.

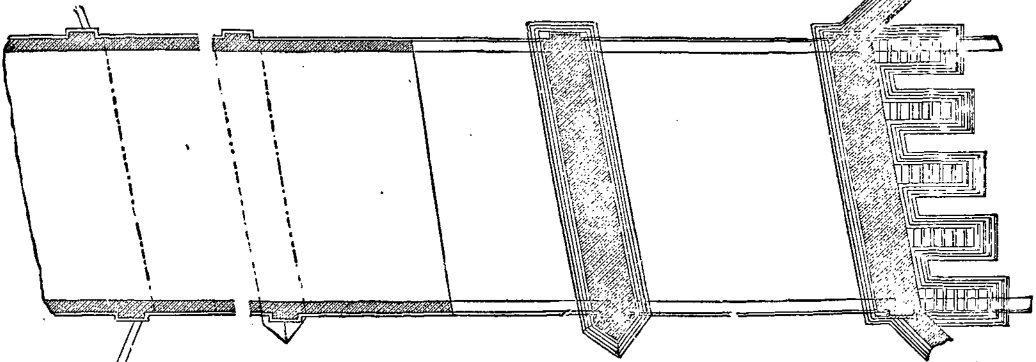


Fig. 1362.

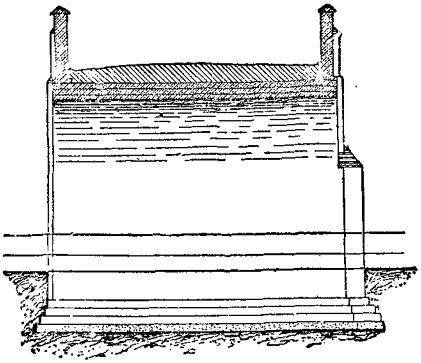


Fig. 1363.

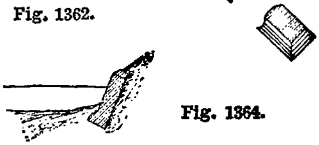


Fig. 1364.

Fig. 1362.—Horizontal Sections of Brick Bridge.
Fig. 1363.—Cross Section of Brick Bridge.
Fig. 1364.—Section through Wing Wall of Bridge.

Brick Bridge, 86-ft. Total Span.

A bridge to be built in red and blue bricks is shown in elevation by Fig. 1360, in longitudinal sectional elevation by Fig. 1361, in sectional plan, through side walls to left of figure and through piers to right of figure, by Fig. 1362, and in transverse section by Fig. 1363. A section of the wing wall is shown in the sketch view (Fig. 1364). The measurement of 86 ft. (approximate) is taken between the two extreme piers.

Curved Wing Wall.

Figs. 1365 and 1366 show the method of finding the elevation of a curved wing wall for a bridge. If made a helical or screw surface at coping level, the wing wall will not be high enough at the centre of slope to fully protect the embankment. It is therefore better to make use of a conic section so that the coping, being a circular quadrant in plan, will show a quarter ellipse in the elevation, and the middle of the curve will then rise higher and cover the embankment. Draw the plan (Fig. 1366) and the elevation of the piers, then draw quadrants as shown by dotted lines, and divide into any number of angles. Draw vertical and horizontal lines from the points on the curves as shown, and the intersections will give points in the curve of the coping. The other lines of the coping may be obtained in a similar manner.

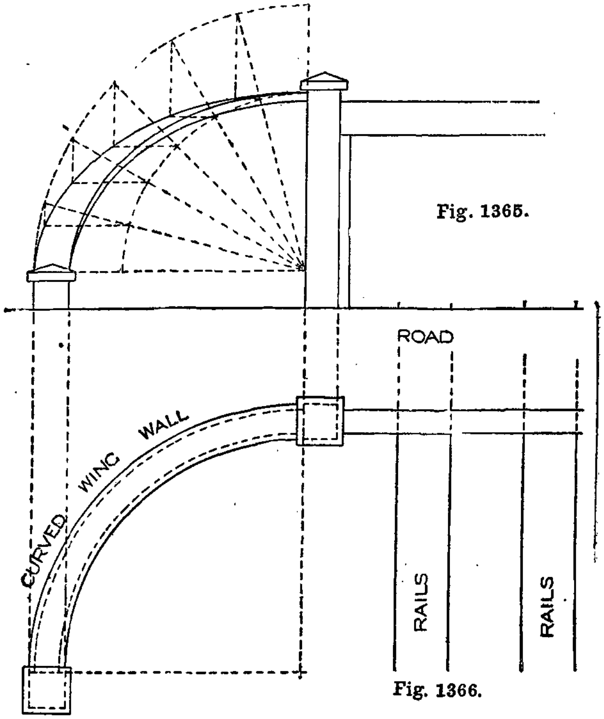


Fig. 1365.

Fig. 1366.

Figs. 1365 and 1366 —Elevation and Plan of Curved Wing Wall to Overbridge.

Masonry Bridges.

A masonry bridge having a public road over it is shown in half elevation and half section by Fig. 1367, plans of the foundations and superstructure being given in Fig. 1368. Fig. 1369 shows a section through the wing wall. A railway bridge forming culvert and cattle arch constructed in masonry is illustrated by Fig. 1370. Fig. 1371 gives a section through the wing wall, Fig. 1372 plans of the superstructure and foundations, Fig. 1373 a cross section, and Fig. 1374 a longitudinal section.

Setting Out Railway Bridge.

The levels are usually given above Ordnance datum, and the levelling commences from the nearest Ordnance bench mark. A stake is usually driven down near the site to a given level for reference during the progress of the work. The centre line of the railway is marked out by pegs, and these give the skew where the railway crosses the road at an angle. The bedstones for girders are fixed at the required level by reference to the stake mentioned above. A general knowledge of levelling and setting out foundations is necessary before beginning work of this kind. Although the contractor has the responsibility, the work is generally set out by the engineer.

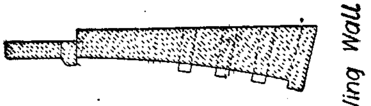


Fig. 1369.

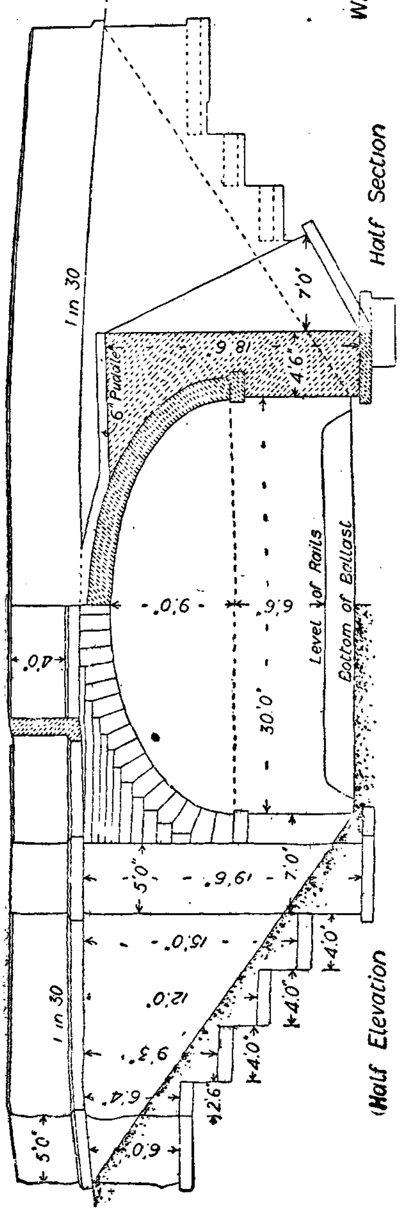

1 in 30
Half Section
7'0"
18'6"
4'6"
6" Puddle
9'0"
6'6"
Level of Rails
Bottom of Ballast
30'0"
4'0"
5'0"
19'6"
7'0"
15'0"
4'0"
12'0"
4'0"
9'3"
4'0"
6'4"
2'6"
6'0"
5'0"
1 in 30
Half Elevation

Fig. 1367.—Half Elevation and Half Section of Masonry Road Bridge.

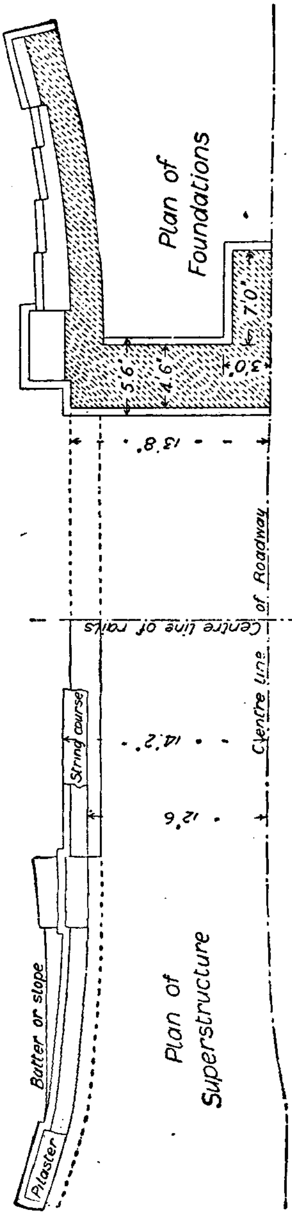

Plan of Foundations
7'0"
3'0"
4'6"
5'6"
13'8"
Centre line of rails
Centre line of Roadway
String course
14'2"
12'6"
Batter or slope
Pilaster
Plan of Superstructure

Fig. 1368.—Half Plan of Superstructure and Half Plan of Foundations of Masonry Road Bridges.

Concrete Bridge.

A roadway is to be carried across a small stream by means of a concrete arch 12 in. in thickness, giving one-sixth of the span for the rise. The concrete is to be 5 to 1; the abutments (of an old bridge now removed) are of masonry. The traffic is very light, the loads, which are carried in small carts, not exceeding 8 cwt. to 10 cwt. Allowance should always be made for heavier traffic that may be reasonably anticipated. The extra cost at the time is not very great, and the result is more satisfactory than if present requirements only had been strictly considered. If the abutments are sound and firm, a concrete arch may be thrown across as shown in Fig. 1375; but if there is any possibility of the abutments spreading, some ½-in. diameter iron rods, 6 ft. or 8 ft. long, should be embedded 2 in. above the soffit of the arch. The line of thrust for an external load of 5 cwt. per ft. super. and the structural load are shown in the illustration, and the maximum compression will amount to about 1·7 tons per square foot. Fig. 1375 shows

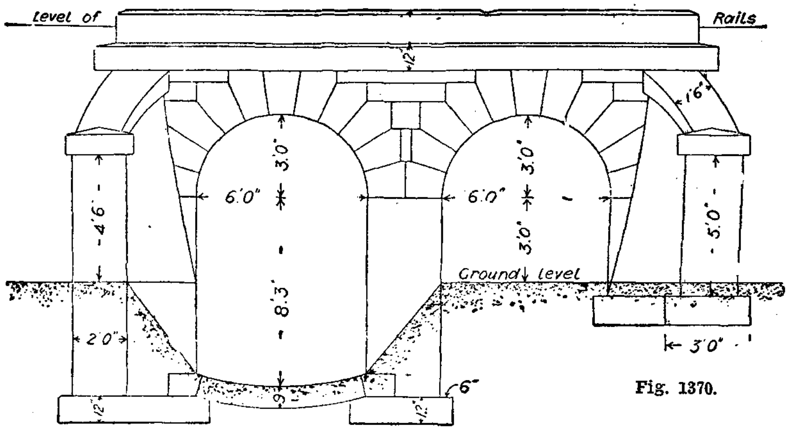

Level of
Rails
12
6"
3'0"
6'0"
3'0"
6'0"
3'0"
4'6"
5'0"
8'3"
Ground level
2'0"
3'0"
6"
9
12
12

Fig. 1370.

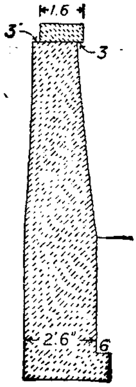


Fig. 1371.

Fig. 1370.—Elevation of Masonry Railway Bridge.

Fig. 1371.—Section through Wing Wall.

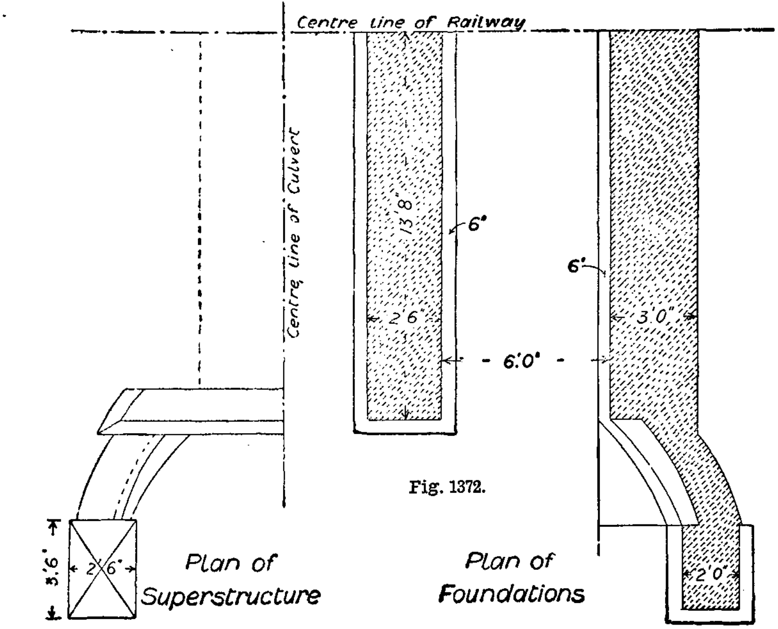

Centre line of Railway
Centre line of Culvert
13'8"
6"
6"
2'6"
6'0"
3'0"
3'6"
2'6"
2'0"
Plan of Superstructure
Plan of Foundations

Fig. 1372.

Fig. 1372.—Part Plan of Superstructure and Part Plan of Foundations.

the arch with its assumed division into voussoirs, and Fig. 1376 shows the reciprocal diagram from which the line of thrust is obtained.

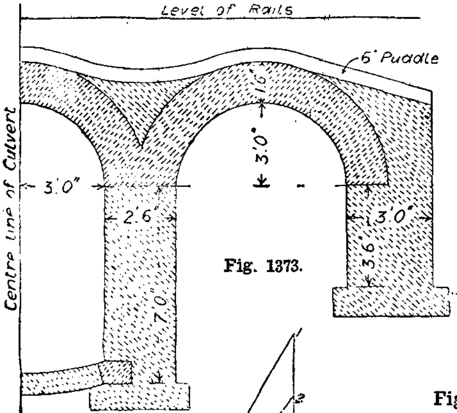

Level of Rails
6" Puddle
1'6"
3'0"
3'0"
2'6"
3'0"
3'6"
7'0"
Centre Line of Culvert

Fig. 1373.

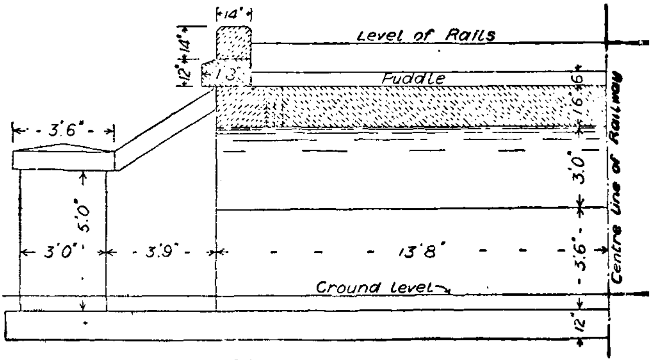

14"
12"
14"
1'3"
Level of Rails
Puddle
3'6"
5'0"
3'0"
3'9"
13'8"
1'6"
3'0"
3'6"
12"
Ground Level
Centre Line of Railway

Fig. 1374.

Figs. 1373 and 1374.—Cross and Longitudinal Sections of Masonry Railway Bridge.

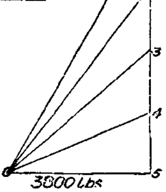

2
3
4
5
3000 Lbs

Fig. 1376.—Reciprocal Diagram for Obtaining Line of Thrust.

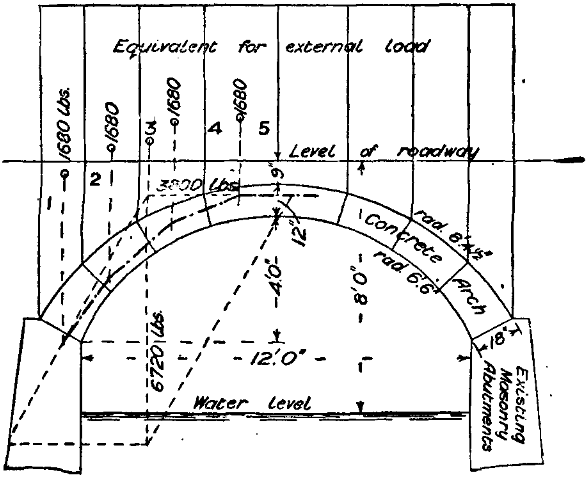

Equivalent for external load
1680 Lbs
1680
1680
1680
1
2
3
4
5
Level of roadway
3000 Lbs
9"
12"
4'0"
8'0"
rad 6'4½"
rad 6'6"
Concrete Arch
18"
6720 Lbs
12'0"
Water Level
Existing Masonry Abutments

Fig. 1375.—Concrete Arch for Roadway Bridge over Stream.

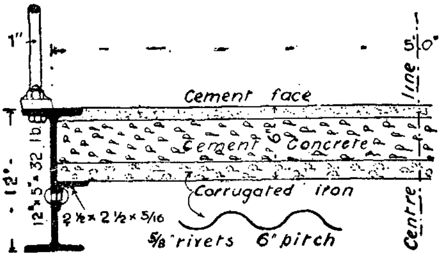

1"
5'0"
line
Cement face
Cement Concrete
6"
Corrugated iron
2½×2½×5/16
5/8" rivets 6" pitch
12"×5"×32 lb.
12"
Centre

Fig. 1377.—Section of Steel Footbridge.

Steel Footbridges.

Footbridge of 20-ft. Span.—A footbridge of 20-ft. span should have girders 1 ft. deep. Assuming the bridge to be 5 ft. wide and carried by two rolled joists with concrete 6 in. thick on angle irons riveted to web and with corrugated galvanised iron underneath to act as centering and be left in, the total load will be, say, 8 tons, or 4 tons on each rolled joist. If 12-in. by 5-in. by 32-lb. steel joists are used they will carry a greater load than is required, but the margin of strength will be useful. A light handrail may be carried by bolting through the top flange, but any holes so made must be completely filled, the bolts being a driving fit. The section of the bridge will be as shown in Fig. 1377. If any horse traffic is possible, another 1½ in. should be added to the thickness of the concrete.

Erecting 70-ft. Steel Footbridge.—In erecting a footbridge (70-ft. span, 8 ft. wide, the girders being two in number, 7 ft. deep, weighing 4 tons each) over a river 4 ft. deep at ordinary times, and 12 ft. at flood, the bridge rising about 6 ft. in the total length, it will be convenient to have the girders sent in halves, delivered on the lower bank and riveted together. A pair of good shear legs should be erected on a temporary platform foundation in the centre of the stream, with a few short piles on the down-stream side to prevent the platform shifting under a freshet. The platform can be kept down by kentledge or old rails or large stones. With a 5-ton crab winch on the higher bank the girders may be picked up singly and slung into position, but the work should not be attempted by inexperienced hands. The lower end of the bridge should have a deep angle iron across the under side to rest against the abutment and prevent it shifting downwards by vibration.

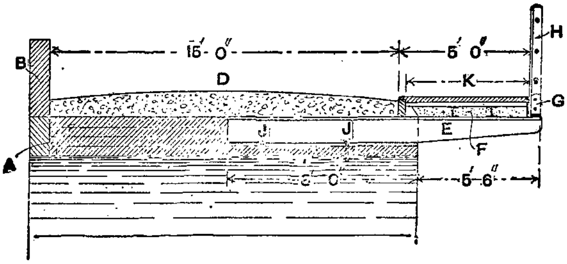
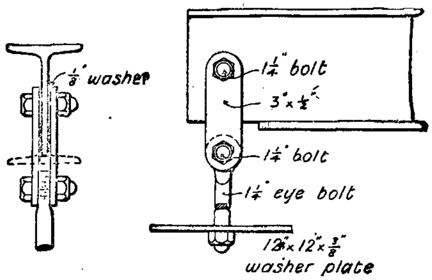


Fig. 1378.—Section of Viaduct showing Footway supported by Cantilevers.



Figs. 1380 and 1381.—Holding-down Bolt on End of Cantilever.

Footway added to Viaduct.

Take a case in which a footway of a certain design is suggested to be added to a viaduct by throwing out steel cantilevers, the work to be done as shown in Figs. 1378 and 1379. Fig. 1378 provides for cantilevers 10 ft. 6 in. apart spaced as shown by the dots in Fig. 1379, and where these dots occur immediately over the arches, the cantilevers are to be bolted through the rings of brickwork with two ⅞-in. bolts; the intermediate joists in the spandrils are to be also bolted down with strong bolts. Over these cantilevers and running from end to end of the bridge rolled iron joists 4 in. by 3 in., filled in with cement concrete, will be placed. The bridge was doubtless designed for the work it is now doing, and there is risk in adding more load to it. The 4-in. by 3-in. joists are rather small, and if joists of that size are used they must weigh not less than 12 lb. per ft. run. Joists of 5 in. by 3 in. by 15 lb. would be better; 10-in. by 6-in. by 45-lb. cantilevers will do, but a stone template about 18 in. square and 6 in. thick should be built in the face of the viaduct under each cantilever. If the stone templates would come too near the outer ring, the pressure may perhaps be spread by using a template of wrought iron 18 in. square and ½ in. thick. The holding-down bolts are of no use near the face of the viaduct, and should be put as far back as possible. A single holding-down bolt at the back end of each cantilever, as shown in the accompanying

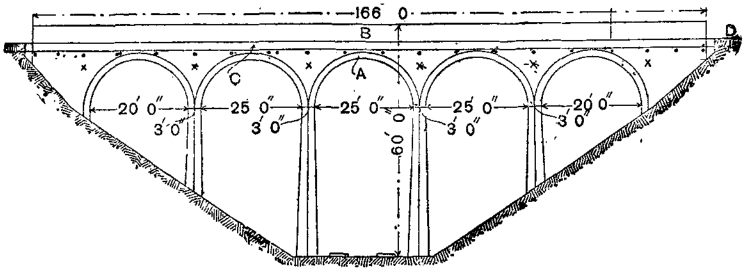


Fig. 1379.—Elevation of Viaduct, showing Positions of Cantilevers for supporting Footway.

illustrations (Figs. 1380 and 1381), would be more efficient. Cleats may be put upon the outer ends of the joists to carry a fascia board, which may have moulded edges or panels formed upon it by simply nailing on mouldings. The letter references to Figs. 1378 and 1379 are :—A, four half-brick rings ; B, parapet 4 ft. high ; C, stringcourse ; D, road line ; E, 10-in. by 6-in. by 45-lb. steel cantilever ; F, Portland cement concrete ; G, 6-in. by 6-in. by ½-in. angle rods ; H, 3-in. by 3-in. by ⅜ in. T iron ; J, ⅝-in. bolts ; K, path.

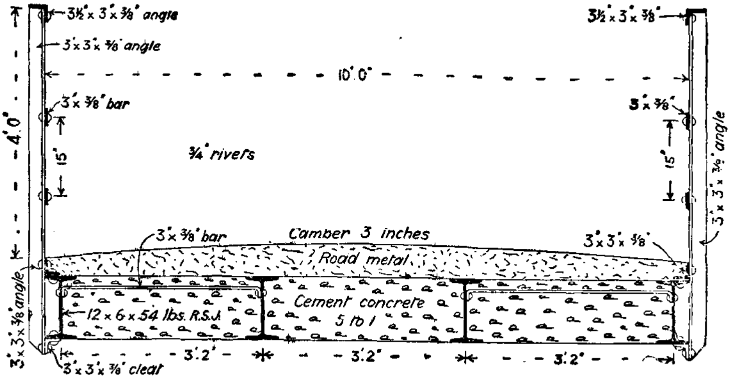


Fig. 1382.—Cross Section of Bridge, 10 Ft. wide, to carry about 6 Tons.

Steel Road Bridges.

21-ft. Span Road Bridge to carry 6 Tons.—Fig. 1382 shows a simple and cheap arrangement of constructing an iron bridge of 21-ft. span and 10 ft. wide, for carrying a weight of a little more than 6 tons. The concrete should be carefully made, and heavy traffic on the bridge should not be allowed for at least one month after the concrete has been laid. The rolled joists should have a bearing of 12 in. at each end on stone templates. The standards and tie-bars should be placed about 4 ft. centre to centre.

Trough Girder Bridge.—In the case of a trough girder bridge of 12-ft. span, with a rolling load of 28 tons on two axles at 8-ft. centres, the form of trough shown by Fig. 1383 would be very bad, the thickness being uniform throughout and the joints all in the top flange. The difference between that and a good form will be seen by comparing Fig. 1383 with Fig. 1384, which is the usual form. In the latter it will be seen that the metal is collected where it is of most value—that is, in the flanges as far as possible from the neutral axis, and the joints are made on the neutral axis between the webs. When the wheel base of a rolling load is greater than half-span, the maximum bending moment will occur with one wheel in the centre of the span, in this case $= \frac{WL}{4} = \frac{14 \times 12}{4} = 42$ ton-feet. The maximum shear stress will occur when one wheel has passed the half-span and the second is just entering upon the bridge. It will then amount to $14 \times \frac{4}{12} + 14 = 18{\cdot}66$ tons under the second wheel. Strictly by the arrangement shown, the stresses will depend somewhat upon the position of the cross sleepers by which the load is applied from the rails to the bridge, but no great difference will result.

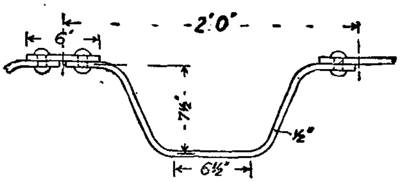


Fig. 1383.

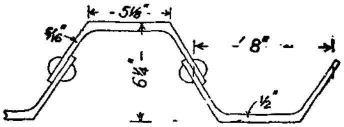


Fig. 1384

Figs. 1383 and 1384.—Bad and Good Forms of Trough Girders.

When the wheel base is less than half span, say 4 ft., the maximum bending moment will be found as follows. The position of maximum bending moment with equal loads will be at $x = \frac{l}{2} \pm \frac{d}{4} = \frac{12}{2} \pm \frac{4}{4} = 6 \pm 1 = 7$ or 5; that is, when one of the wheels is 1 ft. on either side of the centre. The amount will then be $5\,(14 \times \frac{7}{12} + 14 \times \frac{3}{12}) = 58\frac{1}{3}$ ton-feet.

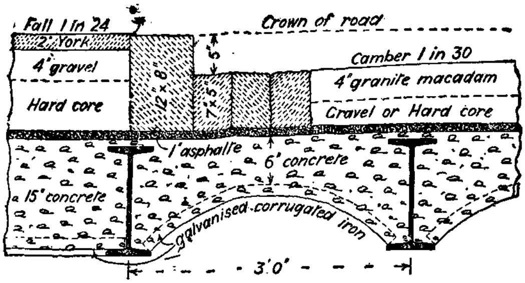


Fig. 1385.—Part Section of Roadway of Steel Girder Bridge.

20-ft. Span Road Bridge.—A road bridge is to be constructed over a stream, the span being 20 ft. clear, width between parapets 35 ft. On a flat platform of rolled steel joists (14 in. by 6 in. by 57 lb., at 3-ft. centres) and concrete, 12 in. of road metal will be spread. The concrete will be 15 in. deep, and a layer (1 in.) of natural asphalt laid with a slight fall from all points to the centre of the bridge, where a proposed pipe passing through the concrete will carry off the surface water to the stream. The parapets (4 ft. high and 18 in. thick) will be constructed of stone. The traffic to be provided for will include a traction engine weighing, say, 15 tons, etc. In getting at the solution to this problem first note that the usual allowance for heavy traffic is 5 cwt. per ft. super. for external load. To this must be added the structural load, which in the present case would amount to $\frac{57}{3}$ = 19 lb. for rolled joists, $140 \times 2\frac{1}{4} = 315$ lb. for concrete and road metal, making together $\frac{19 + 315}{112} = 3$ cwt., or a total of 8 cwt. per ft. super. The allowance of 5 cwt. per ft. super. is sufficient to compensate for the vibration due to live load from traction engine, the actual load of which spread over the full area occupied by the engine will not exceed 3 cwt. per ft. super.; but the load is really not so spread, being concentrated at the wheels. Again, the traction engine, although alone, may be at any part of the bridge, and therefore every part of the bridge must be strong enough for the engine. Some economy can be effected by putting curved corrugated iron plates between the rolled joists in order to reduce the net depth of the concrete to 6 in. in the centre, as shown in Fig. 1385, and thus lessen the weight without reducing the strength. Assuming that the reduced total load is 7 cwt. per ft. super., each rolled joist would have to carry

$$\frac{\text{20-ft. span} \times \text{3-ft. centres} \times \text{7 cwt.}}{\text{20 cwt. in 1 ton}}$$

= 21 tons; for which Dorman, Long & Co.'s *G6 14-in. by 6-in. by 57-lb. rolled steel joist would be just sufficient. The stone parapets, being 4 ft. high and 18 in. thick, will weigh about $\frac{4 \times 15 \times 20 \times 140\text{ lb.}}{112\text{ lb. in 1 cwt.}} = 150$ cwt.; this will be spread over 20 × 2 = 40 sq. ft., giving load of $\frac{150}{40} = 3{\cdot}75$ cwt. per sq. ft., so that a joist similar to the others may be placed on the outside. The two outer bays of rolled joists may be filled with parallel concrete of the full thickness, as they will have less external load, coming under the footpaths, and will then form an abutment for any arch thrust there may be on the intermediate bays. Instead of having the fall on the asphalt to the centre of the bridge with an outlet there to the stream below, the better plan will be to have the fall each way from the centre and an outlet, or outlets, on each side below the parapet walls; the water will thus have less distance to travel, and the bridge will, if anything, be somewhat stronger by the slight arching given transversely. The ends of the rolled joists should rest on good stone templates not less than 2 ft. super. each.

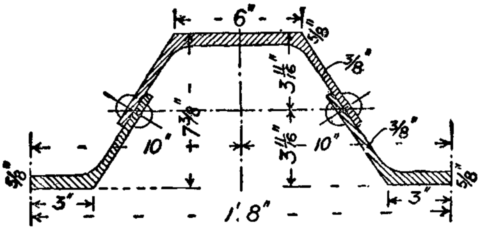


Fig. 1386.—Section of Trough Girder.

and the centre joists can, if desired, be kept a little higher than the side joists. The abutment walls have to hold up the bank of earth at the back, and will need to be at least one-third of the height in thickness and four courses of footings on the water side in 2¼-in. set-offs, the concrete to project, say, 6 in. on each side, and to be not less than 9 in. thick.

Road Bridge to Carry 15-ton Roller.—Although the weight of a 15-ton road roller amounts to only 235 lb. per ft. super. on the enclosing area, the load is a live load, and the equivalent dead load would be 470 lb. per ft. super.; the usual plan, however, is to allow 5 cwt. per ft. super. as provision for all external loads. In addition to providing for this load, the cross girders, or troughing, should be strong enough to carry the concentrated load of 4½ tons on each of the large wheels at 5-ft. 8-in. centres. Although a road roller is only in one place at one time, it will in turn be over every part of the bridge, so that the whole of the bridge floor must be capable of carrying the full load. With the main girders, however, the case is different; they will be strong enough if one road roller on each side of the road in the centre of the bridge is allowed for, and the rest of the bridge is assumed to have an external load of 2 cwt. per ft. super. To all these loads must be added the dead weight of the structure and of the road metal.

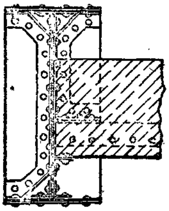


Fig. 1387.

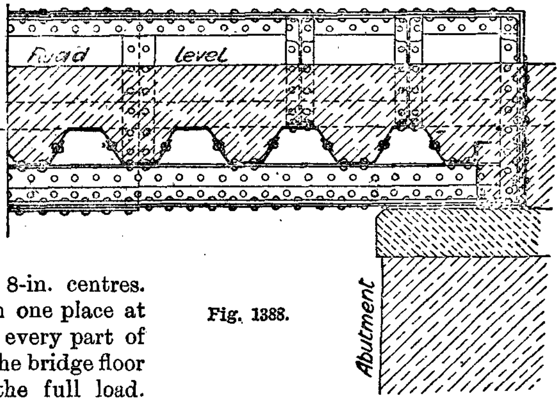


Fig. 1388.

Figs. 1387 and 1388.—Section and Elevation of Part of Road Bridge to carry 15-ton Roller.

46-ft. Span Road Bridge.—A bridge is to have a clear width of 12 ft. and a clear span of 46 ft., and will have to carry a 12½-ton steam-roller. Two main girders and cross girders with arches between, or trough decking instead of the cross girders and arches, will answer. A clear span of 46 ft. would require a depth of 3 ft. to 4 ft. for the main girders. The first step is to determine the character of the road supports. Trough decking will be suitable; the span for it, say, 13 ft. The load will be, say, two wheels carrying 4 tons each at 5 ft. centre to centre, and a distributed load of, say, 1½ cwt. per ft. super. for the weight of the structure and the road metal, and 2 cwt. per ft. super. for the external load. The bending moment from the wheels of the steam-roller will be $4\left(\frac{13-5}{2}\right) \times 12 = 192$ inch-tons. The bending moment from the distributed structural load will be $\frac{1{\cdot}5}{20} \times \frac{13^2}{8} \times 12 = 21{\cdot}26$ inch-tons per foot run of bridge. Dorman, Long and Co.'s trough section "c maximum" (Fig. 1386) has a width centre to centre of 1 ft. 8 in., and a moment of resistance of 198·9 inch-tons. Assuming that the thickness of the road metal is sufficient to spread the load over a trough and half a trough = 2·5 ft., the gross bending moment will be $192 + 21{\cdot}26 \times 2{\cdot}5 = 245{\cdot}15$ inch-tons, and the moment of resistance $198{\cdot}9 \times 1{\cdot}5 = 298{\cdot}3$, which will be on the safe side, but with not too great a margin. The main girders will each have to carry $\frac{13 \times 3{\cdot}5 \times 46}{20 \times 2} = 52{\cdot}32$ tons distributed $+ \frac{1}{2} \times 12\frac{1}{2} = 6{\cdot}25$ tons central, giving a total bending moment of $\frac{52{\cdot}32 \times 46}{8} + \frac{6{\cdot}25 \times 46}{4} = 300{\cdot}84 + 71{\cdot}9 = 372{\cdot}74$ ton-feet. The depth of web being taken as 3 ft., and the allowable stress as 6½ tons per square inch, the required sectional area of each flange in the centre will be $\frac{372{\cdot}74}{3 \times 6{\cdot}5} = 19{\cdot}1$ sq. in. The flanges being made 18 in. wide, the thickness will be $\frac{19{\cdot}1}{18} = 1{\cdot}1$ in., or say one ⅝-in. inner plate, and one ½-in. outer plate, the latter being stopped off half cover length beyond the stress parabola. The complete design will then be as shown in Fig. 1387 (section) and Fig. 1388 (elevation). A brick parapet may be built on the main girder, and the space between the roadway and the top flange can also be filled in. The ends of the bridge may be closed in by a vertical channel bar, riveted to the upturned side of the last trough.

110-ft. Span Road Bridge over River.—A road bridge subject to general traffic should be constructed to carry besides its own weight a distributed load of 5 cwt. per ft. super. A span of 110 ft. and a width of 21 ft. would give $\frac{110 \times 21 \times 51}{20} = 577{\cdot}5$ tons external load. The weight of the roadway and the footpath would be about $\frac{110 \times 22 \times 1}{20} = 121$ tons, and the main girders, say, 75 tons each, making an approximate total of $577{\cdot}5 + 121 + 2 \times 75 =$ say 850 tons to be carried by the main girders, or 425 tons each. Assuming the mean depth to be 8 ft. 6 in., the stress in the flange will be $\frac{425 \times 110}{8 \times 8{\cdot}5} =$ say 688 tons. Allowing 6½ tons per square inch net, this will require $\frac{688}{6{\cdot}5} =$ say 106 sq. in. Flange say 30 in. wide less 4 – 1⅛-in. rivets = 25·5, and $\frac{106}{25{\cdot}5} =$ say 4¼ in. thick. The section for a box girder should now be sketched out and the weight estimated from the dimensions, revising the calculations if the result differs much from the first assumption of 75 tons, and then the complete design should be put in hand.

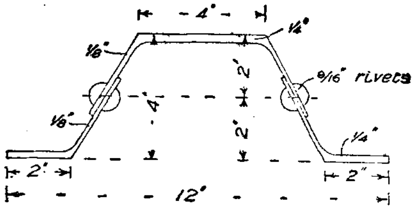


Fig. 1389.—Section of Trough Girder.

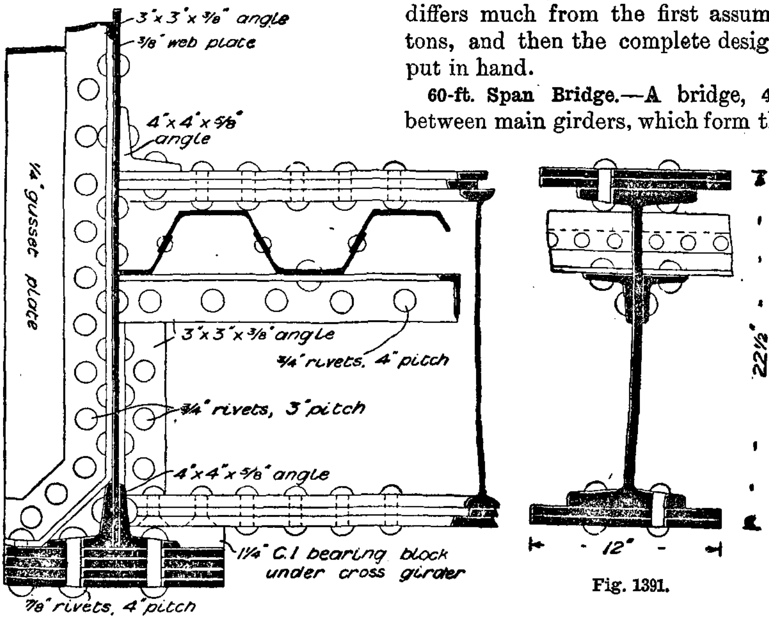


Fig. 1390.

Fig. 1391.

Figs. 1390 and 1391.—Compound Girder, etc., in 40-ft. Span Bridge.

60-ft. Span Bridge.—A bridge, 40 ft. wide between main girders, which form the parapets, is to have cross girders at 10-ft. centres; the total dead and live loads, which may be taken as equally divided, are 3 cwt. per superficial foot of floor space, this being very light for a public road bridge. Trough decking is the usual support for the roadway; but the lightest section made will carry 4¼ cwt. per ft. super. on a 10-ft. span. This is Dorman, Long and Co.'s "section O minimum," as shown in Fig.

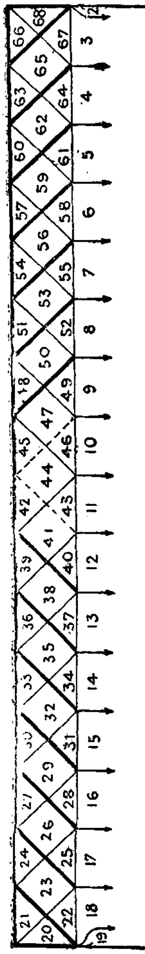


Fig. 1392.—Skeleton Elevation or Frame Diagram of Lattice Girder Footbridge, 80-ft. Span.

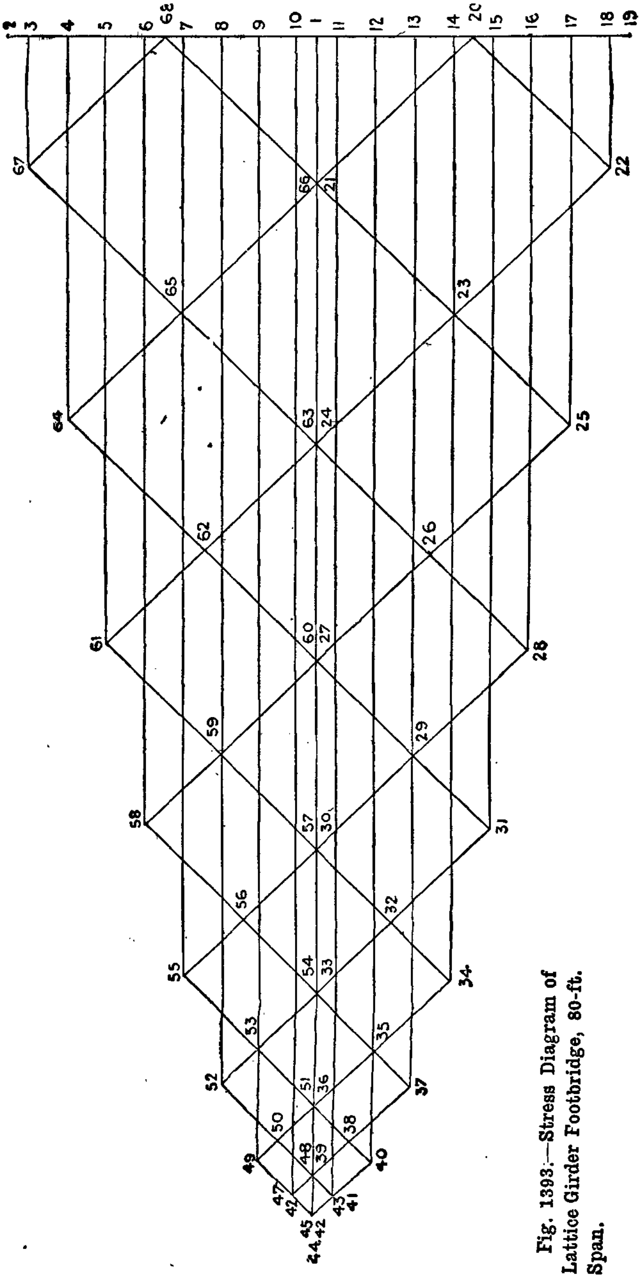


Fig. 1393.—Stress Diagram of Lattice Girder Footbridge, 80-ft. Span.

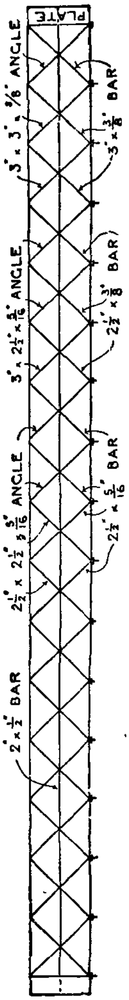


PLATE
3" × 3" × 3/8" ANGLE
3" × 3/8" BAR
3" × 2½" × 5/16" ANGLE
2½" × 3/8" BAR
2½" × 2½" × 5/16" ANGLE
2½" × 5/16" BAR
2" × ½" BAR

Fig. 1394.—Diagram showing Scantlings for Lattice Girder Footbridge, 80-ft. Span.

1389. The cross girders at 10 ft. centre to centre will have to carry $\frac{10 \times 40 \times 3}{20} = 60$ tons, and being 40-ft. span should not be less than 20 in. deep; but a rolled joist of that depth would not be strong enough unless flange plates were added. The lightest section to use would be Dorman, Long and Co.'s compound girder G 1 C 4, but with the outer bottom flange plate increased to $\frac{3}{4}$ in., as in Fig. 1390. The main girders, being, say, 60-ft. span, might be about 5 ft. deep, and if the load brought on by the cross girders be taken as uniformly distributed, the net area of the bottom flange would be $\frac{WL}{8\,df} = \frac{60 \times 40 \times 3}{2 \times 20} \times 60 \div 8 \times 5 \times 6{\cdot}5 = \frac{180 \times 60}{8 \times 5 \times 6{\cdot}5} = 41{\cdot}5$, say 42 sq. in., or say 14 in. by 3 in., made up of four $\frac{3}{4}$-in. plates, with 4-in. by 4-in. by $\frac{5}{8}$-in. angles and $\frac{7}{8}$-in. diameter rivets, 4-in. pitch. The central section would then be approximately as Fig. 1391.

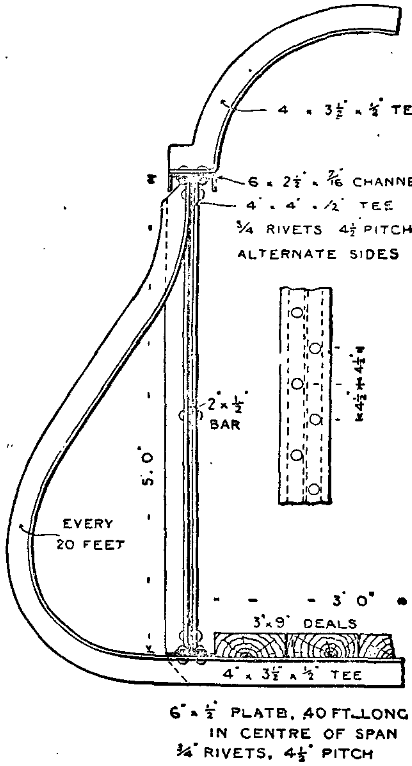

4 × 3½" × ½" TEE
6 × 2½" × 5/16 CHANNEL
4" × 4" × ½" TEE
¾ RIVETS 4½ PITCH
ALTERNATE SIDES
2" × ½" BAR
4½ × 4½
5' 0"
EVERY 20 FEET
3' 0"
3" × 9" DEALS
4" × 3½" × ½" TEE
6" × ½" PLATE, 40 FT. LONG
IN CENTRE OF SPAN
¾" RIVETS, 4½" PITCH

Fig. 1395.—Part Cross Section of Lattice Girder Footbridge.

Lattice Girder Footbridge, 80-ft. Span.

Assume that a lattice girder footbridge, 3 ft. wide, formed with one span 80 ft. long, is to be built across a river. It may be remarked that lattice girders look much lighter than plate girders, and are often of less weight for equal strength. The piers may be of brick or stone, of cast-iron braced columns or of steel lattice-work stanchions. No design of a footbridge that did not allow for a live load of at least 80 lb. per ft. super. over the whole length of the bridge would be compatible with safety; in other words, a crowd of persons must necessarily be allowed for. Span 80 ft., width 3 ft., then $\frac{80 \times 3 \times 80}{2240} = 8{\cdot}6$ tons. The total weight of the bridge will be approximately $\frac{WL}{120} = \frac{8{\cdot}6 \times 80}{120} = 5{\cdot}75$ tons, total, say, 14½ tons, or

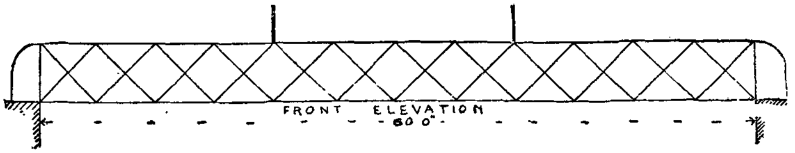

FRONT ELEVATION
60' 0"

Fig. 1396.

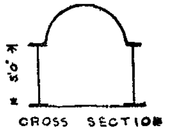

5' 0"
CROSS SECTION

Fig. 1398.

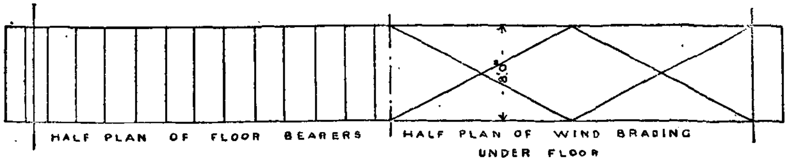

3' 0"
HALF PLAN OF FLOOR BEARERS
HALF PLAN OF WIND BRACING UNDER FLOOR

Fig. 1397.

Figs. 1396 to 1398.—Front Elevation, Half Plans, and Cross Section of Lattice Girder Bridge, 60-ft. Span.

7¼ tons on each girder. Suppose the girders to be 5 ft. deep, there will be 16 bays, and the skeleton elevation or frame diagram will be as shown in Fig. 1392, and the stress diagram as shown in Fig. 1393, from which it will be found that the scantlings of the different parts should be as shown in Figs. 1394 and 1395. It is convenient to give the above stress diagram here, but the stress diagrams for a variety of other steel structures will be given in a later section of this work.

Fig. 1400.

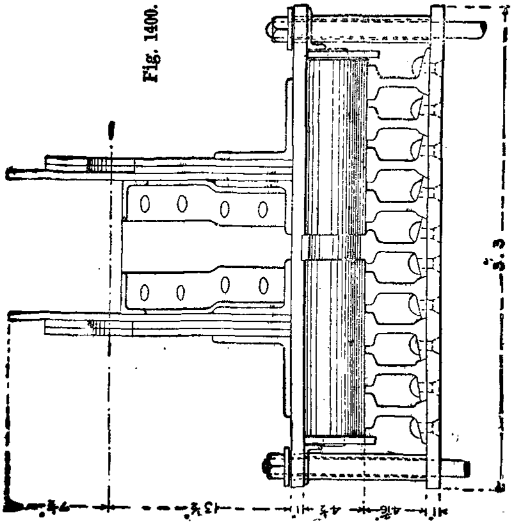
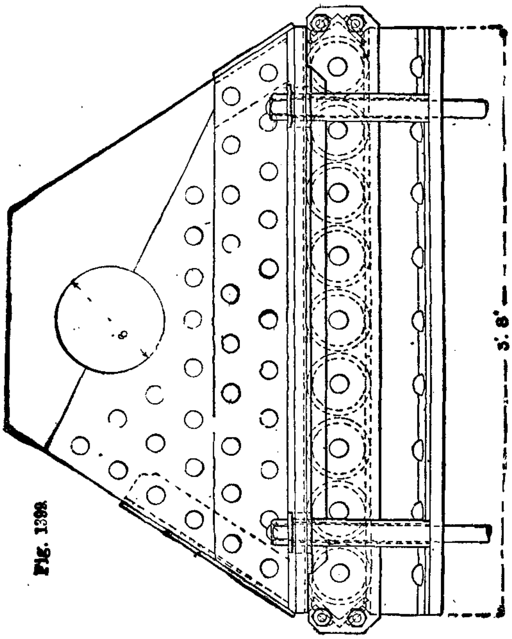


Fig. 1399.



Figs. 1399 and 1400.—Front and End Elevations of Expansion Rollers for Large Bridge.

Similar Bridge, 9 Ft. wide.—The dimensions for a lattice girder footbridge 3 ft. wide are not applicable to a bridge 9 ft. wide, as the structural load and possible external loads will be so much greater; but an approximation may be made by multiplying all the stresses by 3, the ratio of the respective widths, supposing the depth of the girder and the number of bays to remain the same.

Lattice Girder Bridge, 60-ft. Span.

A steel lattice bridge over a river is to have a span between (stone) abutments of 60 ft. and a width of 8 ft.; a rolling load of about 2½ tons is to be carried. In this case the lattice girders should have a depth of not less than 5 ft., making twelve bays of bars at 45° and two vertical pillars of stiffened plate web over the bearing surfaces, as shown in Fig. 1396. The cross girders should be rolled joists or bulb T's spread as shown in Fig. 1397, and the wind ties flat bars under the flooring, as also shown in Fig. 1397. As the load to be carried is not very great, the flanges of the lattice girders will be of comparatively small section, and should be connected over the top at intervals by T sections as shown in Figs. 1396 and 1398.

Bridge Built With Old Rails.

The method described below of bridging a small stream is one of the recognised methods of using up old rails. Assume a span of 9 ft. between abutments, the width of the roadway 18 ft. and a clear height from the bed of the stream to the bottom of the bridge of 3 ft. 6 in., the abutments being 4 ft. thick for the whole width of the roadway. The heaviest load, say, is a 20-ton steam road roller, which, with coals, water, etc., will amount to a rolling load of 25 tons. The proposed method of bridging is to place ordinary second-hand North-Eastern Railway rails, weighing about 80 lb. per yard, side by side for the whole width of the roadway, the rails being held together by three ¾-in. tie-rods; the rails are covered with concrete and road metal. To obtain a definite calculation, the following would be necessary: (*a*) a plan of the road roller with the width of the wheels, distance from centre to centre of the wheels, distance from centre to centre of the axles, and the amount of load on each wheel; (*b*) full-size section of the rail proposed to be used, and whether steel or iron; (*c*) thickness of roadway desired. Whether the rails are bull-head rails or flange rails, the head must stand upwards. The concrete should be made in the ordinary way, 1 Portland cement to, say, 4 or 5 of ballast. Good ashlar stone bearing-blocks would be required under the rails on each abutment.

Expansion Rollers for Large Bridge.

Bridges of large span are generally supported on expansion rollers at one end only, so that one end has a roller bearing and the other end a fixed bearing. The arrangement of the rollers for a bridge 300-ft. span and weighing 150 tons is shown in the accompanying illustrations. Fig. 1399 is the front elevation, and Fig. 1400 the end elevation. The allowance for working load in lb. on each roller per lineal inch is given by the formula

$\sqrt{540{,}000 \times \text{diameter of roller in inches}}$.

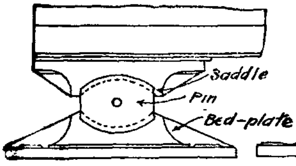


Fig. 1401.

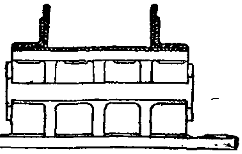


Fig. 1402.

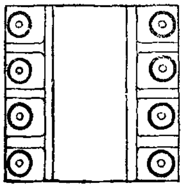


Fig. 1404.

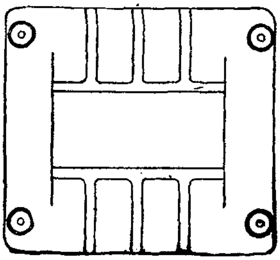


Fig. 1403.

Figs. 1401 and 1402.—Side and End Elevations of Rocker Bearing. Fig. 1403.—Plan of Rocker Bearing Bed-plate. Fig. 1404.—Inverted Plan of Saddle.

Sometimes the bridge is carried on rocker bearings. Figs. 1401 to 1404 show views of a rocker bearing for a large bridge. This is the Horseley bearing, made by the Horseley Company, Ltd., Tipton, Staffordshire.

Suspension Bridges.

Details of a suspension bridge (for foot passengers), 59-ft. span by 3 ft. wide, are here given. The design of a large suspension bridge should be left to a professional expert, but for small spans a contractor accustomed to the work is generally willing to tender for the supply and erection complete without drawings. The general stresses on a suspension bridge are based upon the same principles as the tension in a girder or the thrust in an arch, namely, longitudinal stress equals distributed load × span × 8 times depth or rise or sag. Two types of suspension footbridges are shown by Figs. 1405 and 1406.

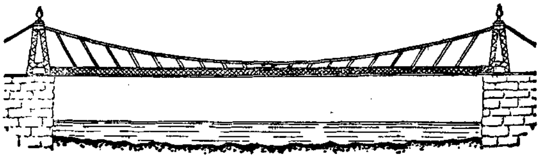


Fig. 1405.—Suspension Bridge.

Suspension Bridge Calculations.—Fig. 1407 shows the outline of a suspension bridge, 96 ft. between the columns. The bridge is carried by the cables, and the roadway does not act as an arch. The load from a crowd = 5 × 96 × 1 = 480 cwt.; the load on each cable = 12 + 5 = 17 tons. Tension in the cable at the lowest point $= \frac{\text{W L}}{8\,d} = \frac{17 \times 96}{8 \times 10} = \frac{102}{5} = 20{\cdot}4$ tons; tension at the highest point $= \sqrt{\left(\frac{\text{W L}}{8\,d}\right)^2 + \left(\frac{\text{W}}{2}\right)^2}$ $\sqrt{416{\cdot}16} + 72{\cdot}25 = \sqrt{488{\cdot}41} = 22{\cdot}1$ tons. Allowing a factor of safety of 10, the breaking weight of the cable should not be less than 221 tons. This would make the cable 4 in. in diameter if a single rope of Bessemer steel were used, or two ropes of 3-in. diameter, or four ropes of 2-in. diameter, or eight ropes of 1½-in diameter for each cable. By the graphic diagram (Fig. 1408), the stress on the column would be 20·5 tons. Being 15 ft. high and 18 in. wide at the top, about 5 sq. in. would be enough

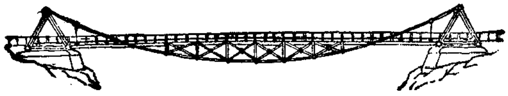


Fig. 1406.—Another Type of Suspension Bridge.

in the four angles to carry the direct load; but allowance must be made for wind and for other unknown stresses due to vibration of the bridge, so that not less than four 3½-in. by 3½-in. by ½-in. angles would probably be required, with bracing about every 18 in. The pull on the anchorage would be about the same as the load on the column, and therefore not less than, say, a 25-ton anchor block of concrete would be required.

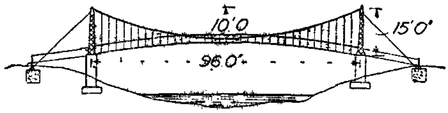


Fig. 1407.—Outline of Suspension Bridge, 96 Ft. between Columns.

Designing Bridge Abutments.

The wing walls add to the stability of a bridge abutment: but this additional strength is not allowed for, as a slight settlement in the foundations of the wing walls would cause a fracture at the junction with the abutment, and the additional strength would be gone. The method of finding the stability is as follows: Fig. 1409 shows a section through the abutment, which is drawn first approximately. Then opposite point A the natural slope of the material forming the embankment is set up, and bisected with the direction of the vertical to give the line of rupture. A line is drawn through the mean direction of the back of the abutment, and the space between this and the line of rupture constitutes the wedge of earth that produces the thrust. Find the centre of gravity of the wedge of earth, and draw a vertical line. At one-third of the height up the back of the wall, draw the line of thrust perpendicularly to the mean direction. At the intersection B of these two lines, set up a height B C equal to the weight of the wedge of earth and the live and dead load on the permanent way over the length of the top of the wedge, divided by the width between the wing walls, being the equivalent total load producing thrust on 1 ft. run of the abutment wall. From the top of this line B C draw a line B D parallel with the line of rupture and cutting the thrust line in D; then B D is the total thrust on 1 ft. run of the abutment. Now find the centre of gravity of the section of the abutment wall above A, and draw a vertical line through the centre of gravity, meeting the line of thrust at E. Below E set off E F equal to the total weight of the abutment above the point A and between the wing walls added to half the total live and dead load on the girders, divided by the width between the wing walls, giving the equivalent total weight for a length of 1 ft. run of the abutment. Produce the thrust line, making E G = B D, and complete the parallelogram E G H F; then E H will be the resultant of the forces. This resultant thrust should fall within the abutment and its foundations, and its effect should be to produce a load per square foot not exceeding 10 tons as a maximum on the outer edge of the brickwork of the top of the footings. The brick footings and the concrete should be of such a width that the line of final thrust E H does not pass beyond the middle third of the total width. This matter will be dealt with more fully in connection with retaining walls.

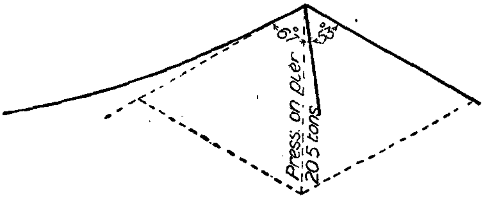


Fig. 1408.—Graphic Diagram for Suspension Bridge.

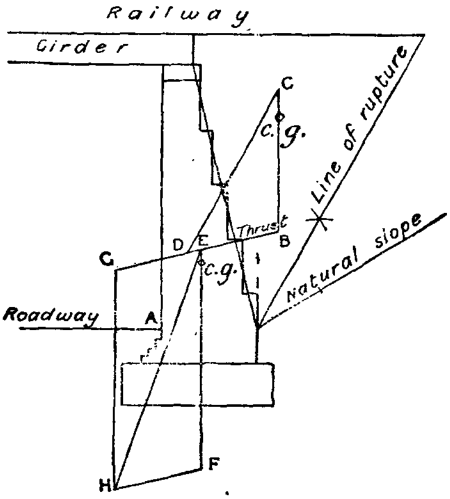


Fig. 1409.—Section through Bridge Abutment, showing Method of Designing.

PLUMBERS' WORK,

DRAINAGE, AND SANITARY FITTINGS.

Manufacture of Lead Pipes.

The principle of the manufacture of lead pipes used in plumbers' work is shown by Fig. 1410. A fire is made under the cylinder, and a flue carried round it to keep the lead just below melting point. A hydraulic ram above the cylinder acts by a cross-head on two pillars attached to an annular piston. The piston has a hole in the centre, and is forced down over a core, causing the lead to be thrown up in the form of a seamless pipe, which as it comes out is coiled on a wooden barrel behind the press. There are also other forms of lead presses. Larger pipes are manufactured from strips of sheet-lead which are first folded round a mandrel and are then soldered at the seams.

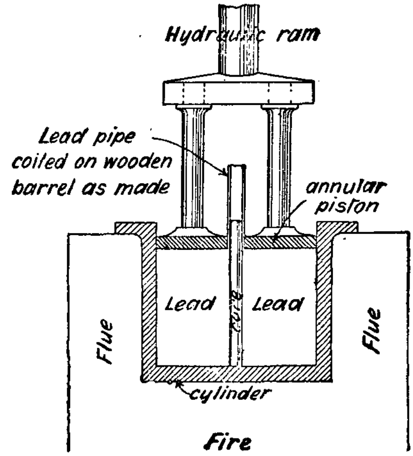
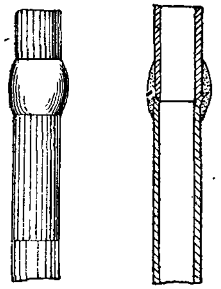
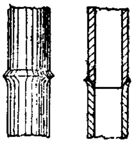


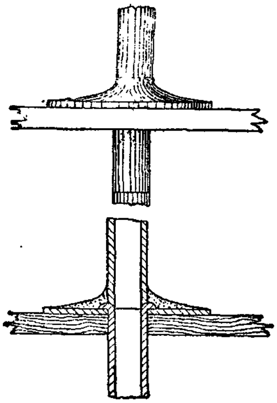
Fig. 1410.—Machine for Making Lead Pipes.



Figs. 1411 and 1412.—Wiped Joint.



Figs. 1413 and 1414.—Blown or Copper-bit Joint.



Figs. 1415 and 1416.—Flanged Joint.

Lead Pipe Joints.

The most usual joint for pipes is the wiped joint (Figs. 1411 and 1412), although, in common work, the blown joint or copper-bit joint (Figs. 1413 and 1414) is sometimes substituted for it. When a vertical pipe has to be supported in its passage through a floor, a flanged joint, as Figs. 1415 and 1416, is used. A flange joint in a 1-in. lead pipe passing through a 1¼-in. floor board is shown in Fig. 1417. A branch or extra joint is made between two pipes, as in Figs. 1418 and 1419. Extra joints are numbered and named by the size of the larger of the two pipes; for example, ¾ in. into 1 in. would be "No. 1 extra soldered joint to 1-in. pipe." A branch joint is generally made in fixing a tap over a sink.

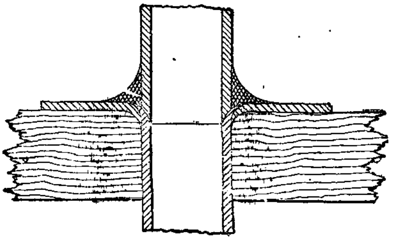
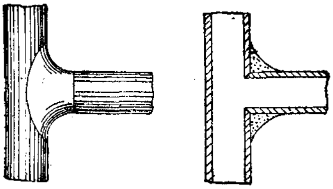


Fig. 1417.—Flange Joint in Pipe passing through Floor Board.

Wiping Joint on Lead Pipe.

To make a wiped joint in, say, a 2-in. upright lead pipe, first cut the ends exactly square, then with a boxwood turnpin open out both pipe ends at the joint, the upper one slightly, the lower more fully. Then rasp or file the ends so that the top pipe has a thin edge inside and the lower pipe has only a slight projection beyond the upper when the latter is inserted to a depth of ¼ in. Shave a short distance inside the lower pipe with a shave hook, and see that the two ends fit closely, previously putting in a piece of cotton waste to prevent the shavings falling down the pipe. The ends of the pipes should then be well chalked and wiped with an old rag or a handful of wood shavings to remove or kill the grease. The ends should next be "soiled" for a length of, say, 4 in. with soil or smudge, a mixture of size, lampblack, and chalk, boiled together with water or stale beer. After they are dry, 2 in. at each end should be shaved to make a clean surface for the solder to alloy with the lead, the cotton-waste should be removed from the lower pipe and the two ends fitted together. Or a piece of paper may be wrapped round the upper pipe for a length of 1¾ in., and round the lower pipe for a length of 1½ in., to avoid soiling and scraping off again. A collar with two ears should then be cut out of 6-lb. lead, so that it may be placed round the pipe on the soiling 3 in. or 4 in. below where the joint is to be, to act as a saucer to catch the solder that falls. It should be well soiled, so as to be removable from the solder, and the ears should be bent over to form a locked joint and hold it on the pipe; or a movable wooden collar may be used. The wiping cloth should be of stout moleskin, fustian, velveteen, or bed-tick. The hot solder or "metal" is put on with a wood or iron splash-stick, or poured from the ladle and wiped round with the cloth to make a regular swelled outline about 3 in. long and 2¾ in. diameter. After completion any of the solder lying over the soiling should be removed by lifting up with a


Figs. 1418 and 1419.—Branch or Extra Joint.

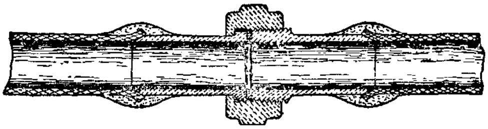


Fig. 1420.—Lead Pipes connected by Brass Union.

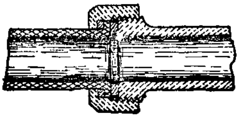


Fig. 1421.—Lead Pipe with Single Union.

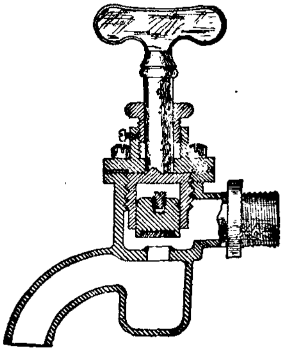


Fig. 1422.—Tylor's Patent Screw-down Bib-cock.

knife. The collar should be removed, and the ring of solder melted on opposite sides with the soldering iron and taken off in two pieces.

Jointing Brass Ferrule to Lead Pipes.

Two methods of jointing a brass ferrule to lead pipes are illustrated by Figs. 1420 and 1421. Fig. 1420 shows two wiped joints and one screwed joint, the latter being formed by the aid of a ring or sleeve threaded internally. Fig. 1421 shows the screwed joint applied directly to the lead pipe.

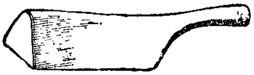


Fig. 1423.—Plumber's Bat or Dresser.

Bib-cock.

A bib-cock is a brass tap with a taper plug having a waterway through it and usually a T head to it for turning on or off, and with the outlet turned down for drawing off liquid. Fig. 1422 shows Tylor's patent waste-not screw-down bib-cock.

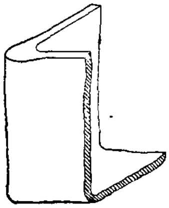


Fig. 1424.—Bossed-up Corner.

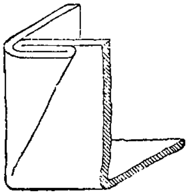


Fig. 1425.—Dog-ear Corner.

Sheet Lead Working.

Various tools are used in dressing sheet lead but the principal one is the bat or dresser (Fig. 1423). The skilful plumber uses the dresser in such a way that the lead is made to flow as required, and in this manner corners of cisterns can be bossed up from the sheet lead as shown in Fig. 1424, instead of being turned over to form a "dog-ear" or "pig-lug" as in Fig. 1425. The latter is more quickly made than the bossed corner.

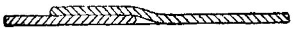


Fig. 1426.—Lap Joint.

Wrinkles in Lead Linings of Pantry Sinks, etc.

The sudden application of hot water to the lead lining of a pantry sink causes a greater expansion of the surface than of the back, and as the two parts are inseparable the front must wrinkle, some of the particles being permanently displaced. Also when the sink is filled with hot water the lead as a whole is expanded considerably, but it is held at various places by copper nails, and it is therefore compelled to bulge. The consequent strain causes a displacement of parts of the lead, leaving it very thin where some of the material has been drawn away, as it never goes back exactly to the same shape.

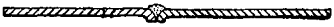


Fig. 1427.—Seam or Soldered Joint.

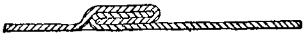


Fig. 1428.—Welt Joint.

Joints, etc., in Lead Roof Work.

The simplest joint used by plumbers is a lap joint (Fig. 1426) from 2 in. to 4 in. or 6 in. wide, according to circumstances. The parts lapping over should be copper nailed, and the joint should only be used for steep slopes or vertical sides. They may also be joined by a seam or plain soldered joint, as in Fig. 1427, or by a welt joint, where the sheets are turned up against each other and dressed over flat, as in Fig. 1428. Where the part to be covered is exposed to the weather, and is wider than one sheet of lead will cover, it is usual to form a roll between the sheets, laid with a fall of 1½ in. in 10 ft., longitudinally. The roll may be solid, as Fig. 1429, which is the method adopted for hips and rolls in general use on roofs, but on flats they may be hollow, as

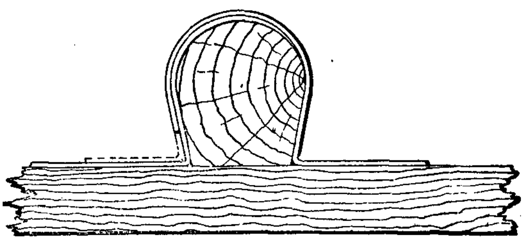


Fig. 1429.—"Solid" Lead Roll.

Fig. 1430, where A and B show the process of formation, but it is not a good form, as they are liable to be damaged. At the ends of the sheets, across the current, it is usual to form a drip, as in Figs. 1431 to 1434. A bottle-nosed drip is so called when the boarding carrying the upper sheet of lead projects over the bearer, the lower sheet of lead being stopped at the under side of the projection and the upper sheet dressed down over the projection and the upper edge of lower sheet, as in Fig. 1435. An improved form of bottle-nosed drip is shown by Fig. 1436. A raglet is a narrow groove, about 1 in. deep, cut in masonry to receive the top edge of an apron flashing, as in Fig. 1437. When the raglet occurs in the top of a blocking course, the process of burning-in is adopted to hold the sheet lead—that is, an under-cut groove is formed for the raglet, and molten lead run in, as in Fig. 1438. When sheet lead is laid on with more slope than is just necessary to permit the water to run off, it is held by soldered dots as in Fig. 1439. The boarding is countersunk, the lead dressed down into the depression, and a screw put through, solder being run over the screw to keep the wet out.

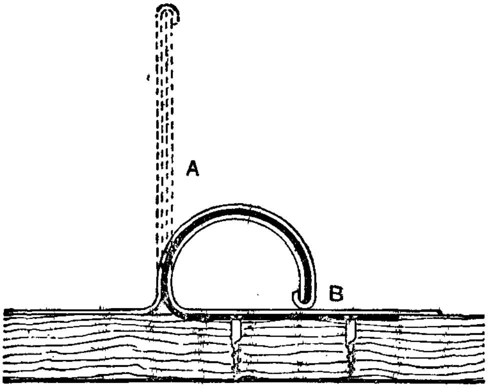


Fig. 1430.—Hollow Lead Roll.

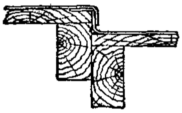


Fig. 1431.—Square Drip.

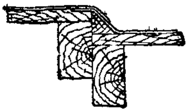


Fig. 1432.—Splayed Drip.

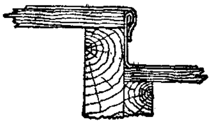


Fig. 1433.—Hollow-nose Drip.

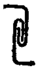


Fig. 1434.—Welted Drip.

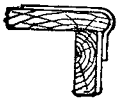


Fig. 1435.—Bottle-nose Drip.

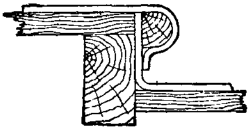


Fig. 1436.—Improved Bottle-nose Drip.

Lead Gutters.

Fig. 1440 represents the cross section through a lead gutter behind a brick parapet, the usual solid roll and splayed drip being used in its construction. The plan of a box gutter is shown by Fig. 1441, and the plan of a central gutter by Fig. 1442, the former being preferable. Two methods of forming secret gutters are illustrated by Figs. 1443 and 1444.

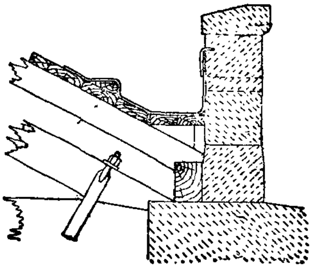


Fig. 1437.—Gutter, showing Raglet in Stone Parapet.

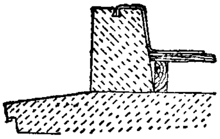


Fig. 1438.—Raglet in Top of Blocking Course.

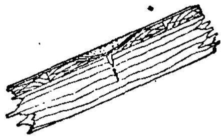


Fig. 1439.—Soldered Dot.

Sizes of Gutters and Rainwater Pipes.

Rainwater pipes and gutters are not usually made the subjects of calculation ; 3-in. pipes and 4-in. gutters for private houses, or 4-in. pipes and 4½-in. to 6-in. gutters for public buildings, are about the average, the down pipes being 30 ft. to 40 ft. apart, but placed wherever it is most convenient to have them. This would seem to make the following a suitable formula:

$$\text{Diameter of pipe} = \sqrt{\frac{\text{roof area square ft.}}{30}}$$

the gutter being fixed with a fall of 1 in. in 10 ft. The slope of the roof does not make any appreciable difference in the result. Putting it another way, a 2-in. pipe would suffice for 120 ft. super. of roof surface, 3-in. for 270 ft., and 4-in. for 480 ft.

Repairs to Lead Gutters.

When the building is fairly old, the treatment must be more thorough than when the gutter alone is in fault. Specification: Take off, sort, and stack for re-use where perfect, all slates and ridge and hip tiles on the side of roof affected. Take off old slating battens and carefully examine all rafters; remove defective or broken rafters except when the foot only is defective, in which case the rafter must be cut back to a purlin, and a scarf made as shown in Fig. 1445, or by means of a vertical or side

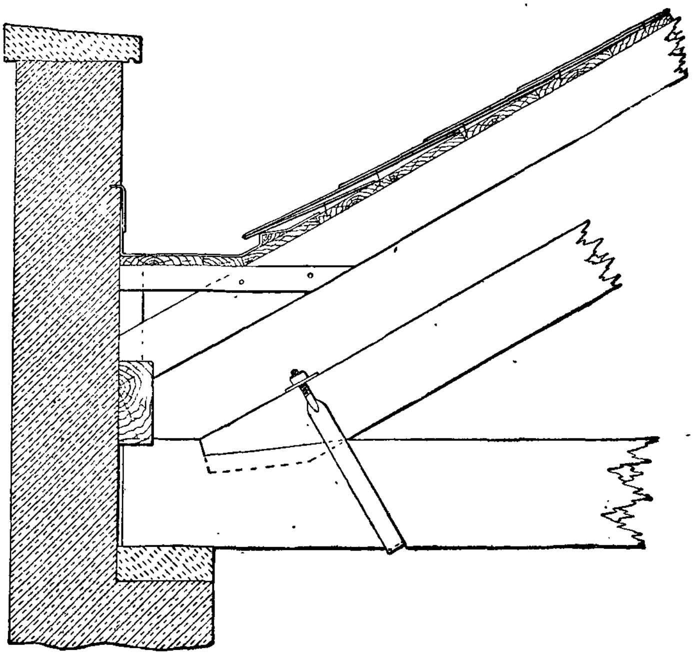


Fig. 1440.—Cross Section through Lead Gutter behind Brick Parapet.

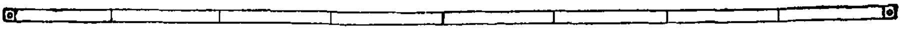


Fig. 1441.—Plan of Box Gutter.

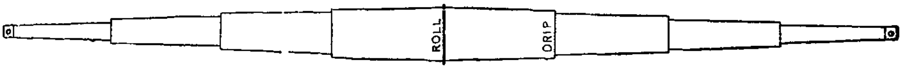


Fig. 1442.—Plan of Central Gutter.

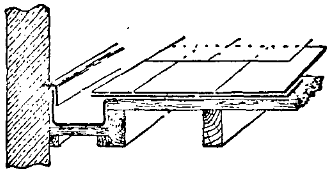


Fig. 1443.—Secret Gutter.

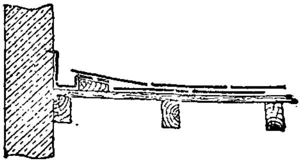


Fig. 1444.—Secret Gutter.

scarf. Any other defective timber in the roof of any kind is to be renewed or repaired in a similar manner, as may be directed after examination of the roof. Form and lay new gutter boards on proper fir bearers, with centre roll, falls, and drips as shown in Fig. 1446 (see also Fig. 1447). Fix layer board on feet of rafters with tilting fillet, and lay all new slating battens. Lay 8-lb. lead gutter, with not less than 4½-in. turn-up at wall and to at least an equal height on the layer board, each fall and drip to be in one length without nailing. Rake out joints in brickwork, and wedge with oak or lead wedges the cover flashing of 5-lb. lead with 4-in. laps. Rake out and point in cement the whole of the exposed surface of brickwork above the gutter. Relay slates and ridge and hip tiles where approved, and make up deficiency with new to match the old. Provide all scaffolding ladders, tools, materials, and water. Clear away all rubbish, and leave perfect at completion.

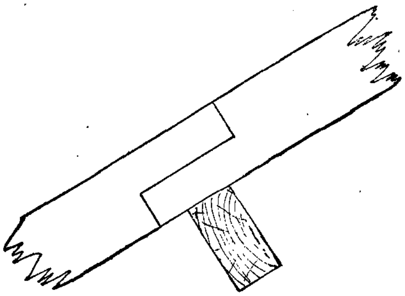


Fig. 1445.—Scarf in Rafter.

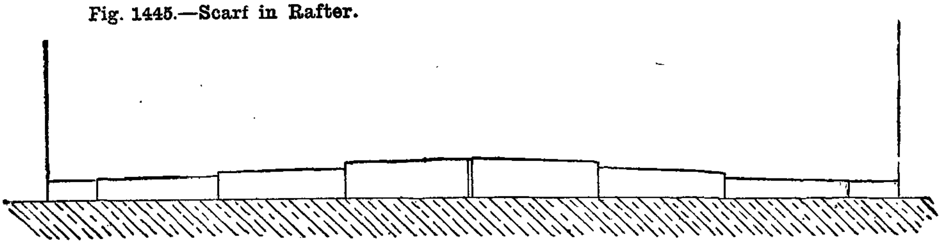


Fig. 1447.—Plan of Gutter.

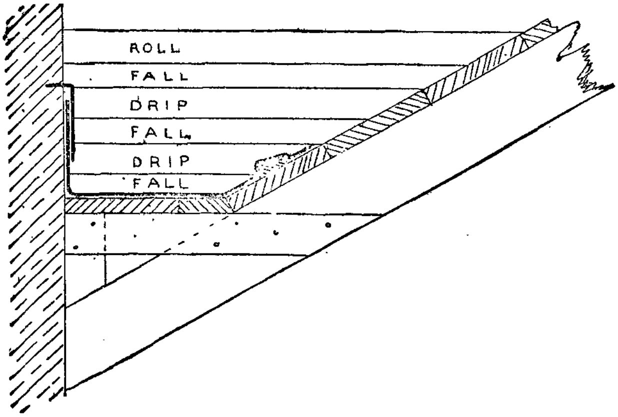


Fig. 1446.—Section of Gutter.

Forming Hips and Valleys in Lead.

Hips and valleys are formed by the junction of roof slopes, the hip being an external angle and the valley an internal angle, both being on a slope somewhat flatter than the pitch of the roof.

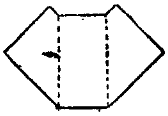


Fig. 1449.—Lead Soaker.

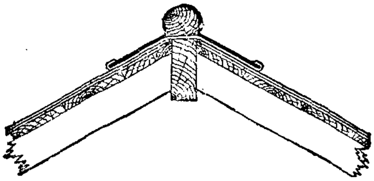


Fig. 1448.—Section of Hip showing Lead Covering and Tacks.

Hips.—Lead tingles or tacks are laid 4 ft. apart across the hip rafter, and the hip roll is spiked over them, then 6-lb. lead, 18 in. or 20 in. wide, and 5 ft. to 7 ft. long, is dressed over the roll well into the angles and nailed at the top end of each sheet under the lap, to prevent sliding down. The tacks are then bent up over the edges of the wings (Fig. 1448) to keep them from blowing up. Another method of covering the hips is to lay soakers, as Fig. 1449, over the hip roll, and work them in under the slating so that they are only visible where they are dressed over the roll as in Fig. 1450. This is the best method for very exposed situations, but takes more lead owing to the number of laps.

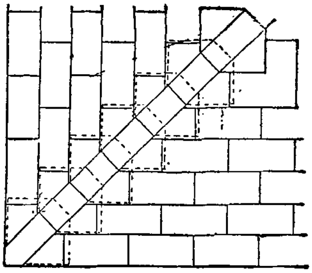


Fig. 1450.—Lead Soakers dressed over Roll.

Valleys.—When a roof is covered with slate or tile battens and not boarded, a layer board must be placed on the slope at each side of the valley rafter to take the lead, which is laid in sheets 12 in. to 15 in. wide and 5 ft. to 7 ft. long. It is dressed down into the angle, and the sides are dressed over tilting fillets and copper nailed. The ends of the sheets lap 4 in. or 5 in. (see Fig. 1451). Soakers may also be used, as in the case of hips.

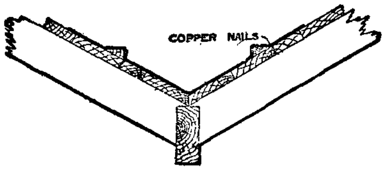


Fig. 1451.—Lead Valley.

Lead Flats.

Fig. 1452 shows a flat roof, covered with lead, size 20 ft. by 16 ft., the figure representing half plan on top, and half plan with boarding removed. Fig. 1453 is a detailed section through flat and walls showing rolled iron joist, bridging joists, firring joists, cesspool, down pipe, etc. A section of the wall, gutter, cesspool, down pipe, etc., is presented by Fig. 1454.

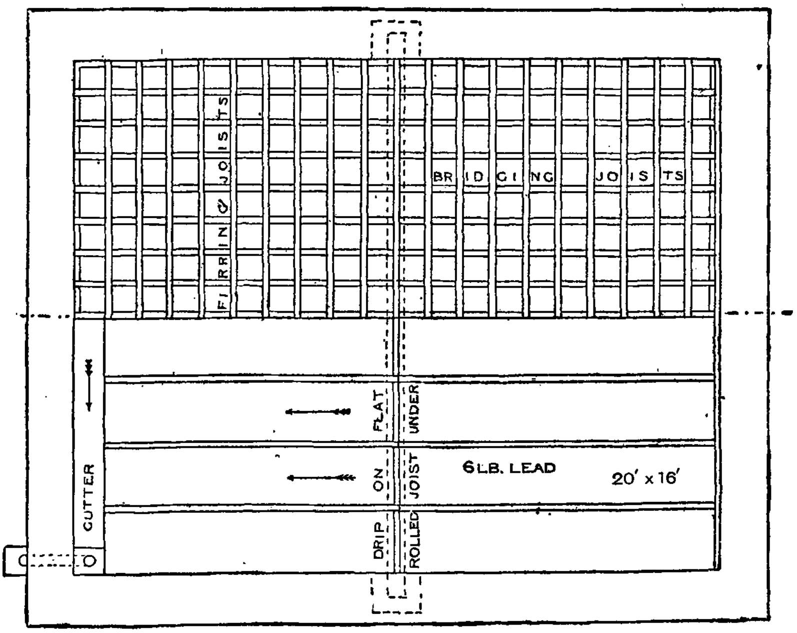
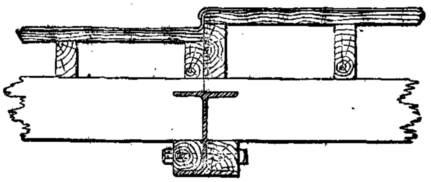


Fig. 1452.—Lead Flat.

Lead Soakers.

A lead soaker is a sheet of, say, 5-lb. lead placed over any part which cannot easily be protected from the weather otherwise. Where ridges stop against the roof plane, as in the case of dormers, a soaker is provided, about 18 in. square. Soakers covered with an apron are sometimes used instead of stepped flashings, one to each course of slates or tiles, worked in between the slates as they are laid, 4 in. to 8 in. under slating or tiling, and 4 in. to 6 in. up wall; this gives not less than 8 in. by 9 in. for each soaker to tiles, and 8 in. by 19 in. for each soaker to countess slates, the lap being 1 in. less for the soakers than for the tiles or slates. Frequently for economy the soakers are made only about 4 in. longer than the gauge of the slating, so that each covers the exposed portion of a slate and laps 4 in. over the next piece. In countess slating it would then be 12 in. or 12½ in. long. Figs. 1455 and 1456 show three soakers with cover flashing in position against a brick wall—slates 24 in. long.



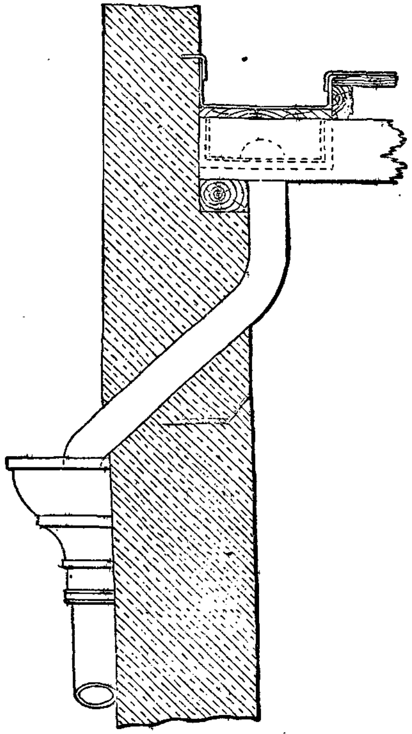


Fig. 1454.—Section of Wall, Gutter, etc.

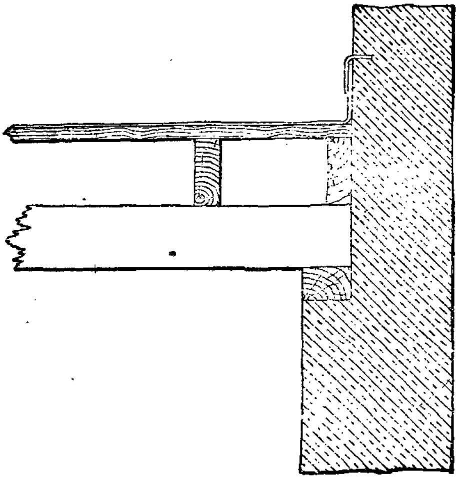


Fig. 1453.—Section through Lead Flat and Wall.

Lead Tacks.

Lead tacks are strips of 5-lb. lead (or often zinc) fixed at intervals to hold down the edge of exposed sheet lead on roofs. In flashings, ridges, and hips, tacks 6 in. by 2 in. are placed 3 ft. 6 in. apart; they are nailed to the boarding or hooked on to the head of a slate and bent over so as to clip the edge of the flashing. In lean-to roofs covered with lead, as the aisle roof of some churches, lead tacks 9 in. by 3 in. are wedged at one end into the wall and soldered at the other on to the lead covering. One tack is taken to each bay formed by the rolls. Lead tacks are also used to secure lead soil-pipe to wall by clipping it round and soldering to it, and nailing at each end, or turning ears over (see Fig. 1457). Tacks are also known as latchets, bale tacks, tingles, and clips.

Lead Work Round Base of Chimney Stack.

Assume that a brick chimney stack, attached to a gable wall or parapet, projects on to a slated roof 1 ft. 10½ in., for a length of 3 ft. 9 in.; the slope of roof is 1 in 2. Figs. 1458 to 1462 show how the roof timbers are trimmed, and how the leadwork should be arranged at the three sides of the chimney stack and against the gable parapet. Take another case. A brick chimney shaft, containing three 14-in. by 9-in. flues, comes up through a slate roof, its longest side parallel to the ridge and about halfway down the slope. Fig. 1463 shows cross section through one of the flues and roof slope. Fig. 1464 shows horizontal section through chimney stack with roof covering removed to show the carpenter's work. Fig. 1465 shows the outside elevation of the chimney stack with step flashing. Fig. 1466 shows plan on roof covering, with back gutter and remainder of plumber's work. Soakers may be laid to each slate at each side of the chimney stack, turned up under stepped apron, or secret gutters may be formed down sides, as alternative arrangements to those shown. Figs. 1467 and 1468 are respectively elevation and cross section through the base of a brick chimney shaft on the outer wall of a building, showing all the details of the lead gutter in rear as well as of the eaves overhanging 18 in., and finished with fascia and soffit boarding and cast-iron ogee gutter. The lead flashings at one end of the shaft are also shown. Zinc is not an economical substitute for lead except in first cost.

Fig. 1455.

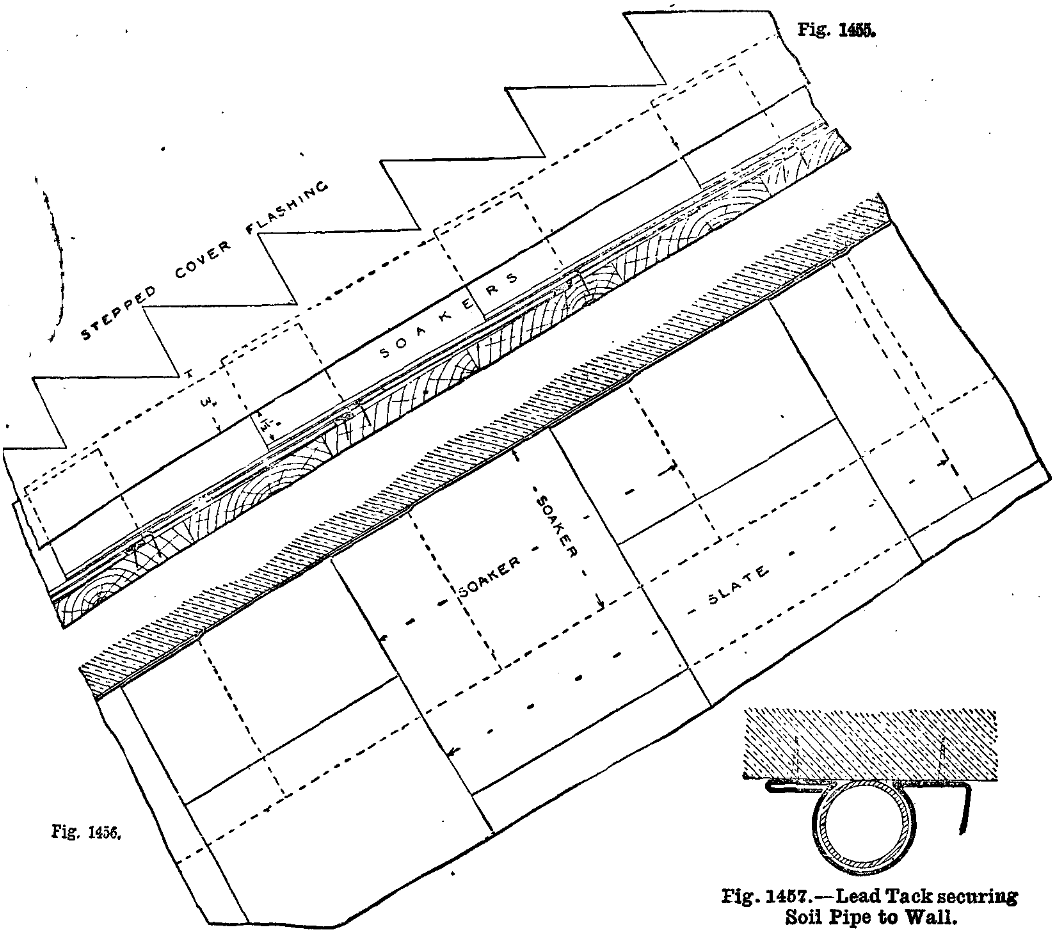
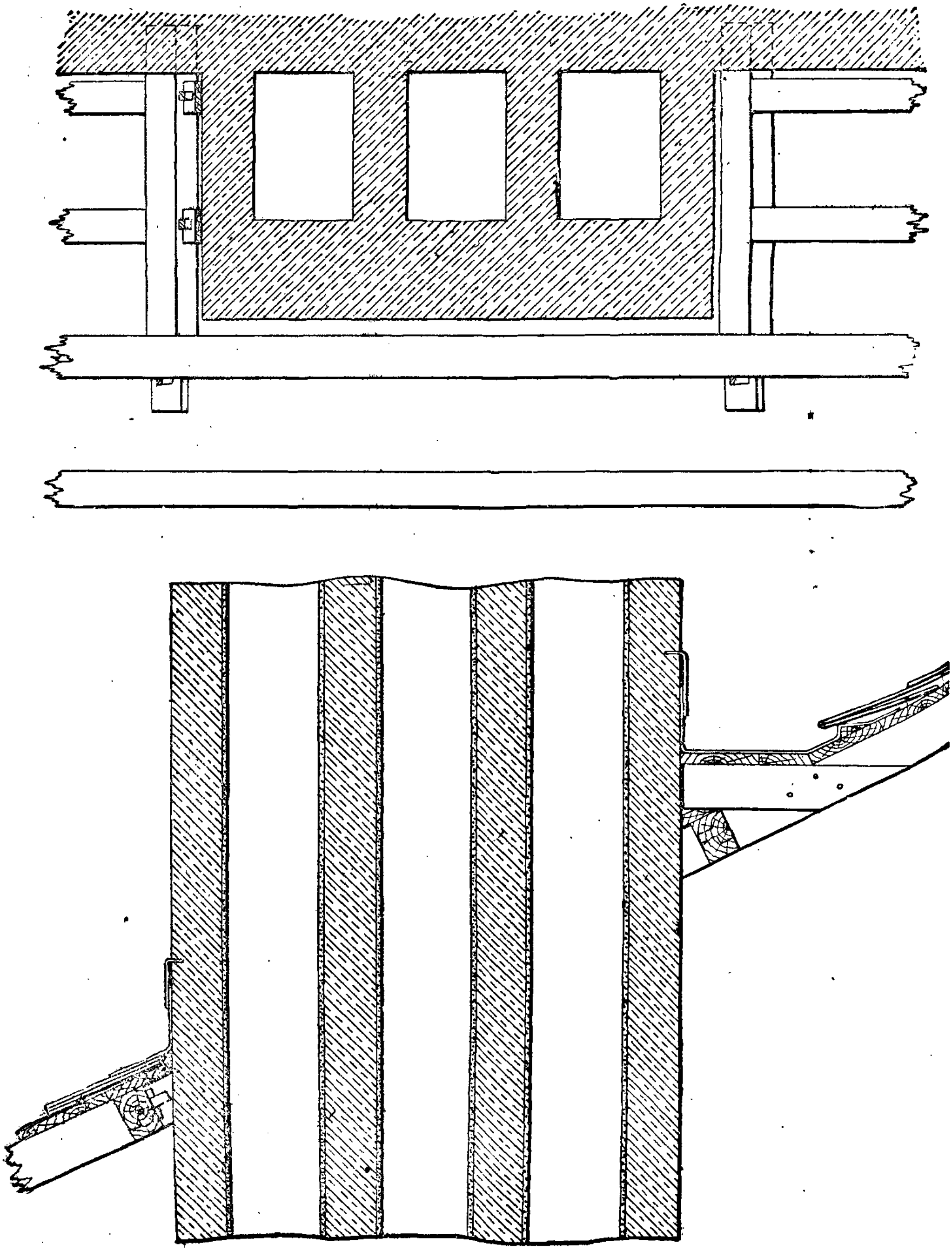


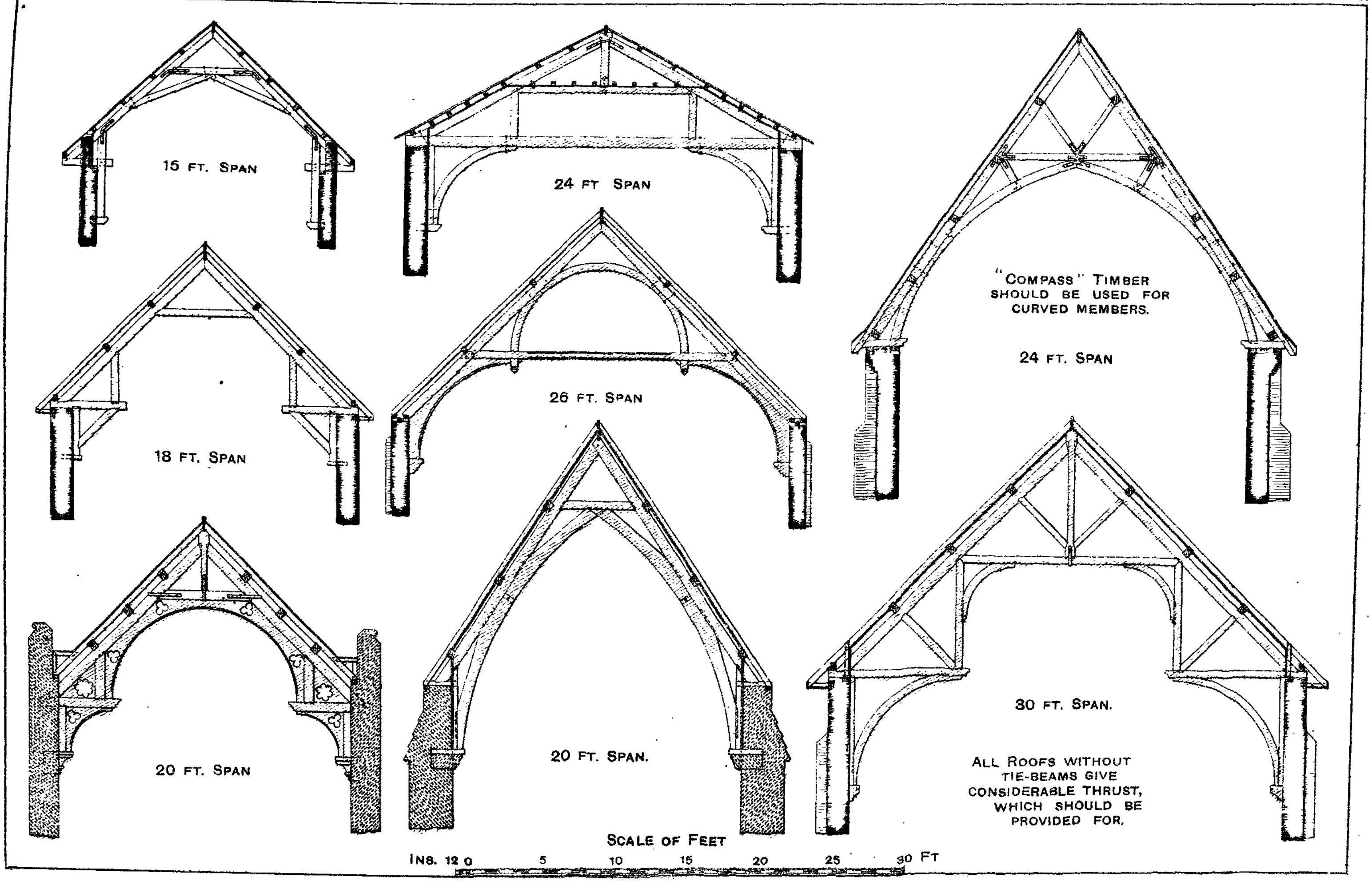
Fig. 1456.

Figs. 1455 and 1456.—Soakers with Cover Flashing in Position against Brick Wall.

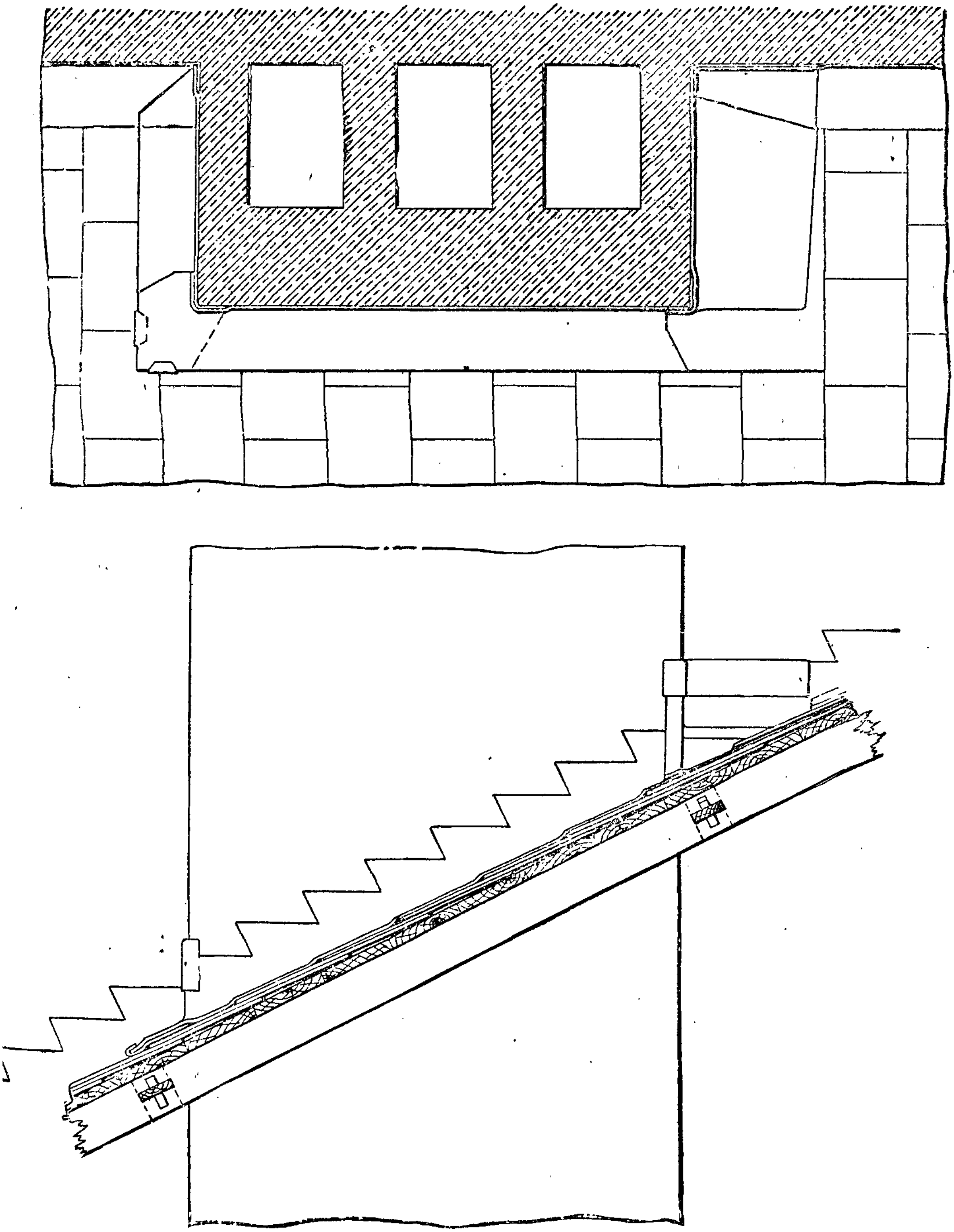
Fig. 1457.—Lead Tack securing Soil Pipe to Wall.



Figs. 1458 and 1459.—Lead Work arranged to Chimney Stack. Scale $\frac{1}{16}$ full size.


15 ft. Span
24 ft Span
"Compass" Timber should be used for curved members.
24 ft. Span
18 ft. Span
26 ft. Span
20 ft. Span
20 ft. Span.
30 ft. Span.
All Roofs without tie-beams give considerable thrust, which should be provided for.
Scale of Feet
Ins. 12 0
5
10
15
20
25
30 Ft

OPEN TIMBER ROOFS.



Figs. 1460 and 1461.—Plumbers' Work round Chimney Stack.

Covering Right Circular Cone.

A right circular cone, to be covered with 6-lb. lead, is 3 ft. in diameter at the base and 2 ft. high. Assuming that the joints are butted, it is required to find the weight of the lead. The surface area at the sides of a right cone, 3 ft. diameter and 2 ft. high, will be circumference of base multiplied by half slant height $= 3 \times 3{\cdot}1416 \times {\cdot}5 \times \sqrt{1{\cdot}5^2 + 2^2} = 11{\cdot}771$ sq. ft.; taking this as the area of the 6-lb. lead, the covering will weigh 70·68 lb., say 71 lb. Fig. 1469 shows the development of the sheet lead.

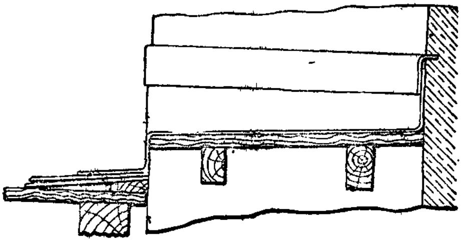


Fig. 1462.—Section showing Lead at Upper Side of Chimney Stack.

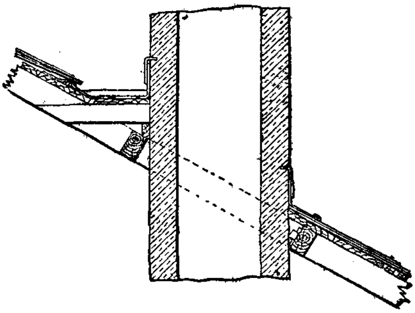


Fig. 1463.

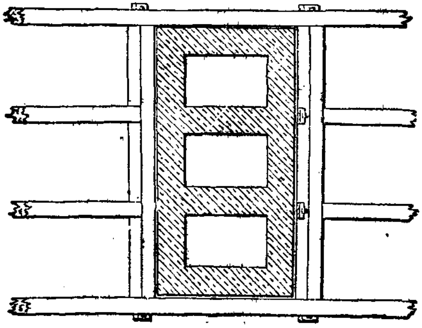


Fig. 1464.

Another Case.—A right circular cone roof, over a lantern, is 8 ft. in diameter at the eaves and 3 ft. high (from base at eaves to vertex), and it is to be covered with copper plates in the manner of slating; they are trimmed and bent to fit on. The margin is 6 in., the lap is 3 in.; the apex is finished with a conical cap of sheet copper, 8 in. diameter at the base; the sheet copper weighs 3 lb. per superficial foot, omitting nails; it is required to find the total weight of copper on the roof. Circumference of base of cone $= d\,\pi = 8 \times 3{\cdot}1416 = 25{\cdot}1328$, the vertical height is 3 ft., therefore the slant height will be $\sqrt{4^2 + 3^2} = \sqrt{16 + 9} = 5$ ft., and the surface area $= \dfrac{25{\cdot}1328 \times 5}{2} = 62{\cdot}832$ sq. ft. The margin shown by the copper plates is 6 in.; there will therefore be 10 rows, laid as slates; each exposed portion of 6 in. by the

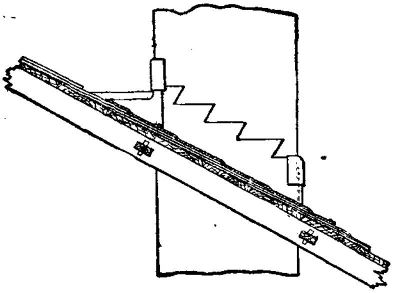


Fig. 1465.

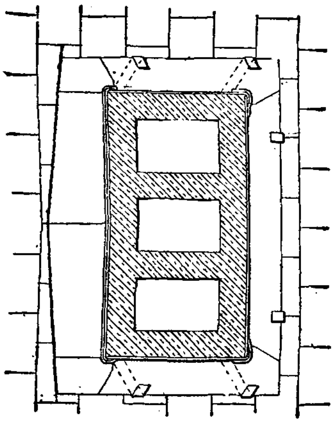
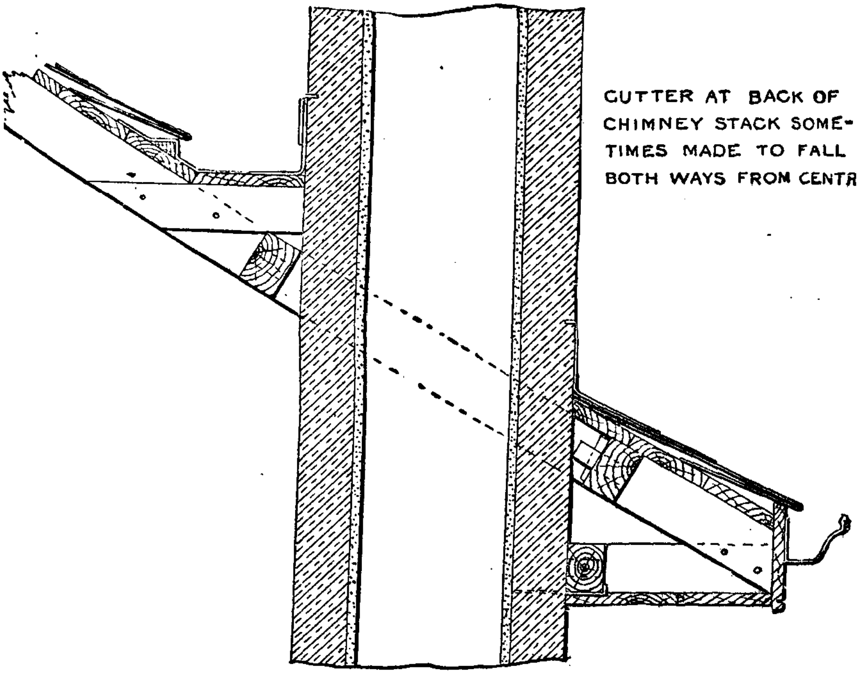
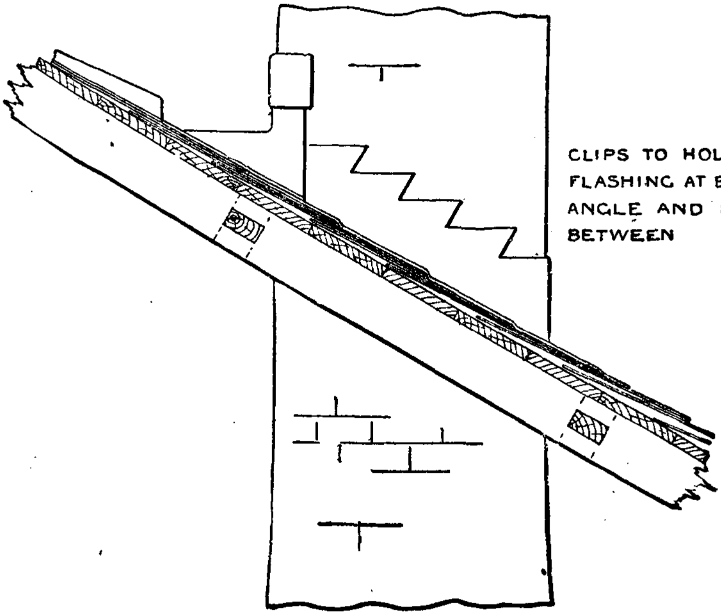


Fig. 1466.

Figs. 1463 to 1466.—Lead round Three-flue Chimney Stack, passing through Slate Roof.



GUTTER AT BACK OF CHIMNEY STACK SOMETIMES MADE TO FALL BOTH WAYS FROM CENTRE



CLIPS TO HOLD DOWN FLASHING AT EACH ANGLE AND MIDWAY BETWEEN

Figs. 1467 and 1468.--Lead Work around Brick Chimney Shaft, showing details.

width will represent 2½ times the amount, owing to the double thickness and the lap in addition, or a weight of $62.832 \times 2\frac{1}{2} \times 3 = 471.24$ lb. The conical cap is 8 in. diameter, and will therefore be 3 in. high, the surface area $= \frac{8 \times 3.1416 \times 5}{2 \times 144} = .436$ sq. ft., or $.436 \times 3 = 1.308$ lb., which added to the weight of the copper plates $= 471.24 + 1.308 = 472.548$, say 472½ lb.

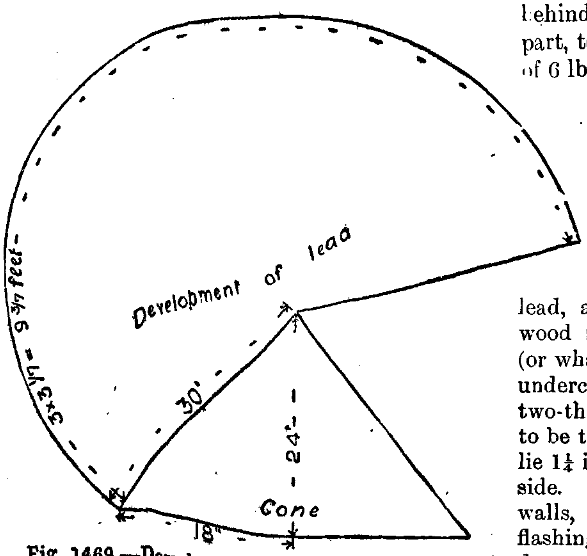


Fig. 1469.—Development of Sheet Lead for covering Right Circular Cone.

Specification for Plumber's Work on Roof of a First-Class Dwelling-House.

The work described in this specification is to include all solder, copper nails, oak and lead wedges, wall hooks, tacks, lead collars, etc., necessary to make the construction perfect and complete. The sheet lead is to be made of soft pig-lead milled to an even thickness, well and evenly dressed without injury to the surface; to be of the specified weight in all cases, and to be weighed when required at the contractor's expense, from sample selected. The flats and gutters to be all laid to fall, to have a minimum fall towards outlet of 1½ in. in 10 ft., to have not more than 10 ft. between drips. Except in special cases, and with the architect's express sanction, the work is generally to be done with ⅛ sheets 11 ft. by 3 ft. 6 in., then to be secured to one another by rolls, with bossed ends and drips. No solder joints to be used. All nailing to be done with copper nails. Lapped joints for flashings, ridges, etc., to lap not less than 4 in. Form the lead gutters to main roof, where shown on drawings, with 7-lb. lead (9 in. in narrowest part), turned up by 9 in. under slates and dressed over 3 in. tilting fillet. The lead to be turned up 6 in. against parapet wall, and covered by an apron of 5-lb. lead, inserted 1½ in. into front of brickwork (or groove in masonry), and turned down 4 in. over flashing. The apron secured to wall with lead wedges, and the joint pointed in cement. The gutters behind chimneys to be 6 in. wide at narrowest part, to slope both ways from centre, and to be of 6 lb. lead, dressed over tilting fillet and 6 in. beneath slates, the end turned up 5 in. against wall, and dressed 3 in. round return of chimney covered with apron of 5-lb. lead, dressed down 4 in. over flashings, and stepped down in brick courses over soakers on return of chimney. Cover the wood flats shown on drawings with 7-lb. lead, and carefully dress the lead over the wood rolls, which are to be 1¾ in. by 1½ in (or whatever size may be determined) and well undercut, the undercloak being made to cover two-thirds of the wood roll, and the overcloak to be turned right round the roll and made to lie 1¼ in. on the lead flat on the least exposed side. The lead to stand up 6 in. against all walls, and flashed with 5-lb. lead cover flashing 6 in. wide. The drips to be 2 in. deep. The cover flashings are to be secured with 6-in. by 2½-in. tacks of 6-lb. lead. The ridges and hips to be secured by 9-in. by 2½-in. 7-lb. lead tacks. Soakers to be of 4-lb. lead, one to each slate, and to lie 5 in. under slates, and turn up 5 in. against wall, with stepped flashing fixed over them. Put slates of 7-lb. lead where ends of iron pipes or stays pass through roof, properly bossed and dressed round pipes, and to have the full lap of slating. Form in gutters to main roof where marked on plan a cesspool of 7-lb. lead, not less than 9 in. square and 6 in. deep, the lead bossed to shape of cesspool, dished, and rebated to receive tafted end of 4-in. drawn lead pipe of 7-lb. lead, bent to necessary curve to connect cesspool with head of rainwater pipe; cover cesspool with copper-wire dome. The lead pipe, where it passes through wall, to be covered with tarred felt. Flanks of dormer where they abut against roof to have secret gutter formed of 5-lb. lead, dressed over tilting fillet and 8 in. under slates, and turned up 5 in. against side of dormer (slates to finish at least 3 in. from side of dormer). The dormer cheeks to be covered with 6-lb. lead, turned over and close copper-nailed along vertical side and top, and to have secret tacks where necessary for supporting the lead. This covering to overhang the flashing of secret gutter 4 in., and to be cut to the rake of the roof and turned round front with wide ear and dressed over oak sill, and copper-nailed in water-drip of same as details, or where roof abuts against side of dormer lead soakers are to be used, as described previously, the lead covering of dormer cheeks dressed down over them. Form the gutter between the two roofs as shown on plan, and cover same with 7-lb. lead, dressed 6 in. up vertical sides, level with tilting fillet, with proper drips as before described, put apron of 5-lb. lead dressed over tilting fillet, taken 6 in. under slates and turned down to within 1 in. of surface of gutter. Cover the shaped and rounded rolls of ridges with 6-lb. lead, dressed well into the angles under the wall, and lapped 7 in. over the slates on each side with 6-in. lapped joints. Cover the hips with 6-lb. lead in a similar manner, but each sheet copper-nailed at the upper end. Lead of hips and ridges to be secured with lead tacks, as described previously. The valleys of main roof to be covered with 6-lb. lead 18 in. wide, dressed to slope of boarding, turned over tilting fillet on either side, and dressed 6 in. under slates, and to have lapped joints not less than 6 in. deep.

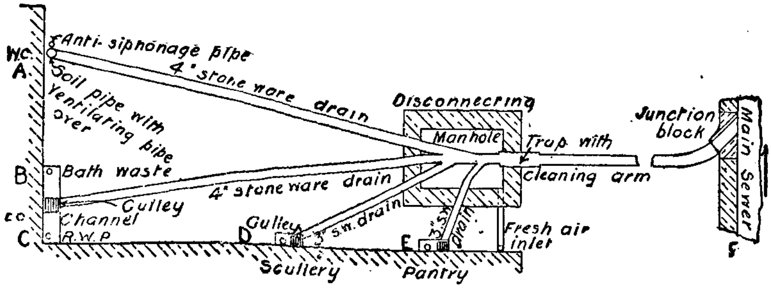


Fig. 1471.—House Drainage System.

Average Depth of Sewers.

The level of a sewer is fixed by the level of the outfall and the gradient required to suit the sectional area. When the difference of level is insufficient, flushing or pumping will be required. Twenty feet is no uncommon depth for a main sewer. The average depth of a sewer trench is easily found when the levels of the two ends are given, by adding the two end depths together and halving the sum to obtain the mean depth. When a sewer is to cross under a railway at right angles, or nearly so, in the natural undisturbed gravel soil, if great care be used in bedding the iron pipes solid, the depth may be as little as 2 ft. between the rail level and the top of the pipes. The filling-in should be very carefully rammed at the sides to make all compact and solid. In clay soil the ramming must be aided by watering.

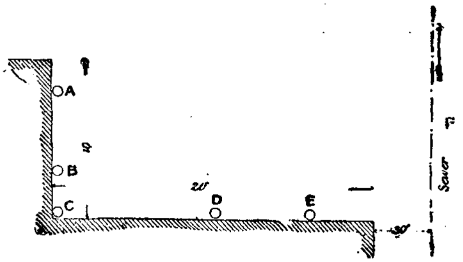


Fig. 1470.—Outline of Building and Direction of Sewer.

Measuring Fall in Drain.

Assume that it is desired to determine the fall of a drain. In a case where the sewer is only 3 ft. below the surface of the road, and the back premises are some 30 ft. away and below the road level, there might not be fall enough for a drain to cleanse itself without a flushing tank. A 4-in. drain should have a fall of 1 ft. in 40 ft. to 48 ft. The best way of getting the levels in this case would be with a spirit level on a straightedge, packed up on bricks and wooden wedges, from the point in the road where the depth of the sewer is known to the back gully. Then the difference of level from the outlet of the gully to the springing line of the sewer will give the total fall, which, divided into the distance, will give the rate of the fall. Thus, difference of level 18 in., distance 30 ft., rate of fall $= \frac{30 \times 12}{18} = 1$ in 20, but by keeping the pipes at a gradient of 1 in 40 there would be a margin for a drop at the disconnecting trap, according to the pattern used.

House Drainage System.

The whole duty of house drains is to carry away all excreta and liquid refuse with the least danger to health. Fig. 1470 shows the outline of a portion of a building and the direction of a public sewer, F, near it; A indicating water-closet soil pipe; B, bath waste; C, downspouting; D, scullery waste; and E, pantry waste. The proposed arrangement of drainage is shown in Fig. 1471, the channels to gullies being shown in detail at Fig. 1472, and the disconnecting manhole at Fig. 1473. Fig. 1474 shows how the soil pipe at the back of a house in a terrace should be ventilated and connected up with the street sewer.

Drainage System for Houses in Row.

Fig. 1475 shows how to carry the drainage of a house in a row, where it has to pass under the basement floor, towards the sewer in the street.

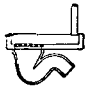


Fig. 1472.—Channels to Gullies.

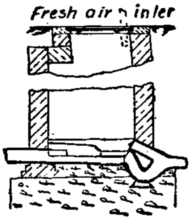


Fig. 1473. —Disconnecting Manhole.

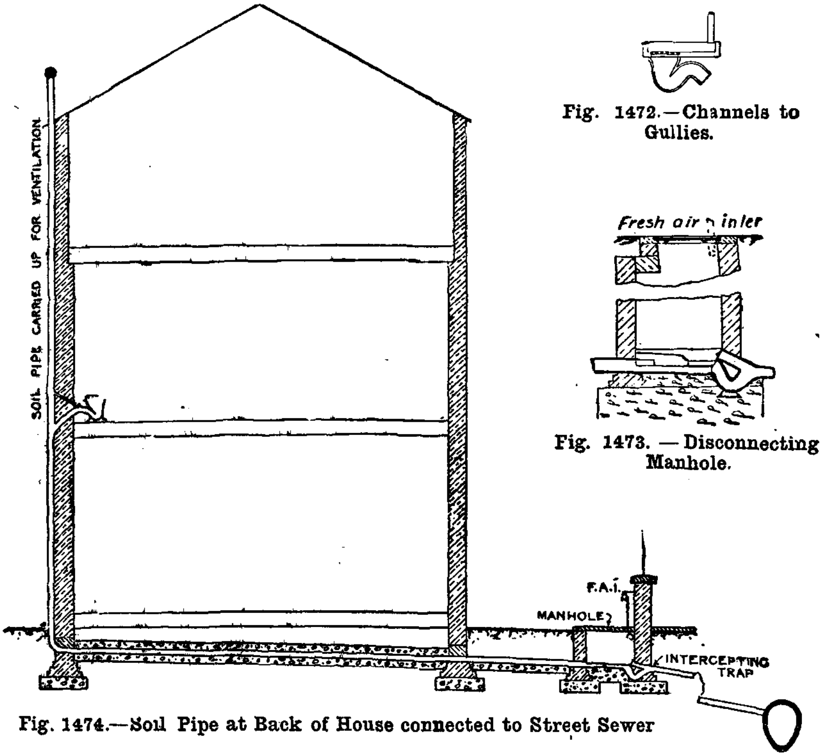


Fig. 1474.—Soil Pipe at Back of House connected to Street Sewer

In Fig. 1475, S P indicates the soil pipe, and A S I the anti-siphonage pipe, carried up from the trap of the lower w.c. to such a height that the open end is not less than 3 ft. above the level of any window within 20 ft. of it. In the back area the R W P discharges into a stoneware channel, terminating in gully G. The scullery sink discharges into the gully, the waste pipe being of drawn lead, trapped and provided with a screw cleaning eye. The front area is constructed with a fall to the gullies G G, the gullies being connected with the main drain within the inspection manhole I M. The manhole is covered with an air-tight cover and provided with a fresh-air inlet F A I. A disconnecting trap D T is fixed at the outgo of the manhole. The inspecting arm of the trap inside the chamber is closed with a movable stopper, so fixed and bedded as to be air tight. The pipes are glazed stoneware socketed pipes, embedded in concrete 6 in. thick round every part of the pipe. If the pipes have to be kept 3 ft. above the basement floor, they should be of cast iron and carried on brick piers, the joints run with lead and carefully caulked.

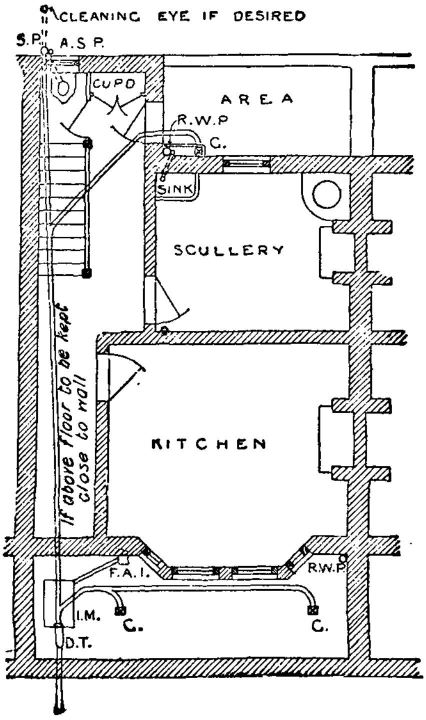


Fig. 1475.—Drainage System of House in Row.

Jointing Stoneware Socketed Pipes.

In jointing the pipes, the back portion of the socket is filled with hemp gasket for a depth of ½ in., and the remainder of the socket is filled with Portland cement, gauged with an equal volume of clean sharp sand, and a fillet of the same composition is put round the whole of the front of the socket, flush with the outer edge, and finished off at an angle of 45° to the pipe.

Jointing Stoneware Pipe to Lead Pipe.

The joint at the foot of the lead soil-pipe is made by means of a brass ferrule. This ferrule is connected to the lead pipe by means of a wiped joint, and is inserted in the stoneware socket, the joint being made with Portland cement, as described for the ordinary joints.

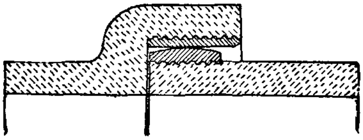


Fig. 1476.—Section of Doulton's Double Seal Joint.

Drain Testing.

To test the drain for leakage at joints before the ground is filled in, a drain stopper should be inserted at the lower end in the manhole, and water poured in at the furthest gully trap, or at the cleaning eye at end of drain, the gully trap being then plugged with clay. If the water remains at the same level at the bend for one hour, the pipes are tight. If the water level goes down, search must be made for the point of escape. Each branch drain with separate connection to manhole must be tested independently. After all is completed, and the earth is filled in and rammed, the smoke test may be used. The pipe from a smoke machine may be passed through the seal of the gully, and the whole system filled with smoke until it appears at the top of the ventilating pipe, when that should be closed with a wet cloth, and pressure increased until the smoke returns through the seal of the gully. A thorough search should then be made along the whole course of the pipes for any appearance of smoke, which would thus indicate the possibility of foul air escaping. Usually the smoke

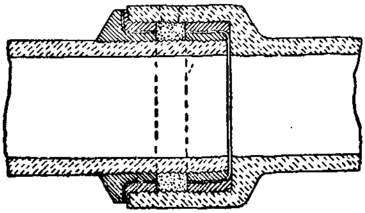


Fig. 1477.—Section of Hassall's Double Seal Joint.

is passed into the drain at its entrance to the manhole, and the test does not include the manhole cover joint, fresh-air inlet, etc.

Double Seal Joints for Drain Pipes.

Figs. 1476 and 1477 show double seal joints which have been successfully used for keeping subsoil water out of drains; but, of course, any good joint, such as the ordinary plain socket and Portland cement, will answer if the foundation is firm so that the pipes cannot shift. Stanford's joint is shown by Fig. 1478.

Cast-Iron Brackets for supporting Sewer Pipe.

Brackets to be used for supporting sewer pipes may be of the design shown in Fig. 1479. The curve of the pipe seat in the bracket should be say ⅛ in. larger radius than the pipes, so as to avoid risk of jamming at the entrance. All angles should be filleted. The lewis bolts need not be more than 1 in. in diameter, but they should be let well into the wall. One bracket to each pipe will be sufficient.

◆

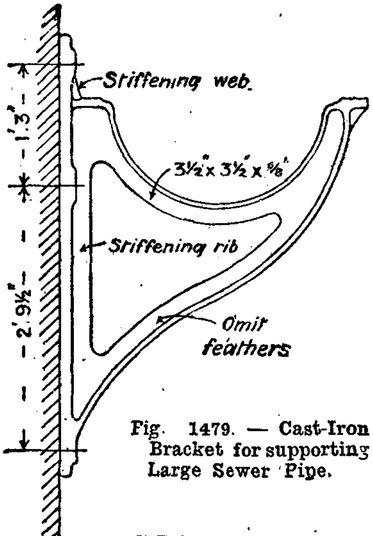


Fig. 1479. — Cast-Iron Bracket for supporting Large Sewer Pipe.

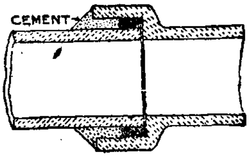


Fig. 1478.—Stanford's Drain Pipe Joint.

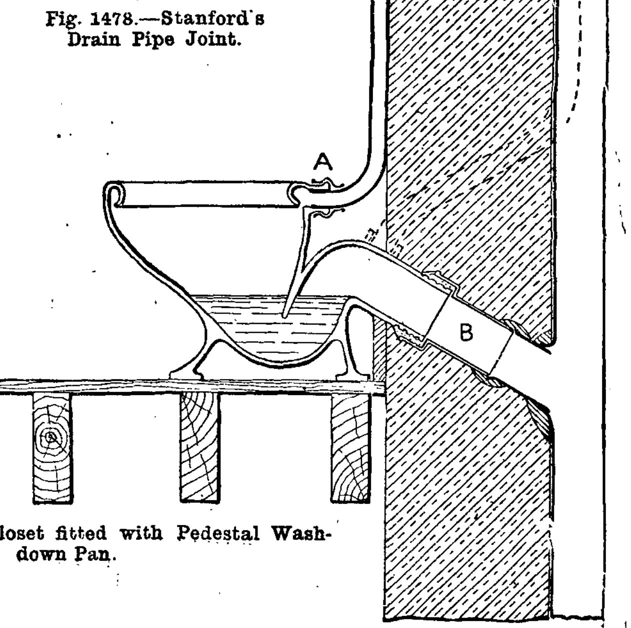


Fig. 1480.—Water Closet fitted with Pedestal Wash-down Pan.

Typical Drainage Job.

Below is described the whole business of opening a flagged yard; sinking a trench; laying 9-in. glazed stoneware spigot and socket pipes jointed with Portland cement mortar; refilling the trench and reinstating the flagged yard (the ground being easy ground, timbering is not required). The position of the trench is marked on the surface of the flags with two chalk lines, and an open joint is looked for. The flag is then tapped round with a beetle, or wooden rammer, or the end of a piece of timber, to loosen it, and the point of a pick is inserted in the joint to lift it. The others are then readily lifted without damage, and they are all stacked against the wall for re-use. If numbered before being taken up, so much the better. The width of trench will vary with the depth, but 2 ft. to 2 ft. 6 in. will probably be sufficient. The material being thrown out on one or both sides, the excavation is made to the required depth and to the proper fall,

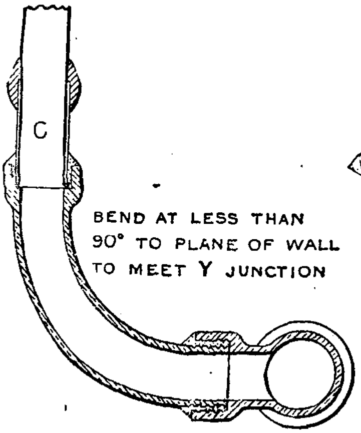

C
BEND AT LESS THAN
90° TO PLANE OF WALL
TO MEET Y JUNCTION

Fig. 1481.—Stoneware Bend to connect Soil Pipe to Drain.

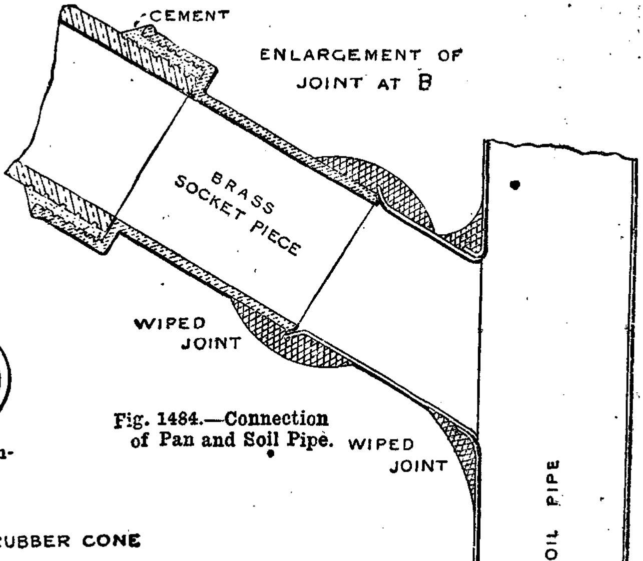

CEMENT
ENLARGEMENT OF
JOINT AT B
BRASS
SOCKET PIECE
WIPED
JOINT
WIPED
JOINT
LEAD SOIL PIPE

Fig. 1484.—Connection of Pan and Soil Pipe.

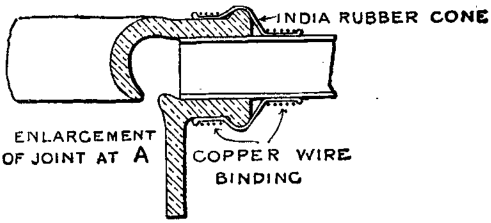

INDIA RUBBER CONE
ENLARGEMENT
OF JOINT AT A
COPPER WIRE
BINDING

Fig. 1483.—Pan Rim Jointed to Flushing Pipe.

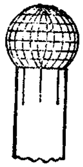


Fig. 1482.—Top End of Soil Pipe.

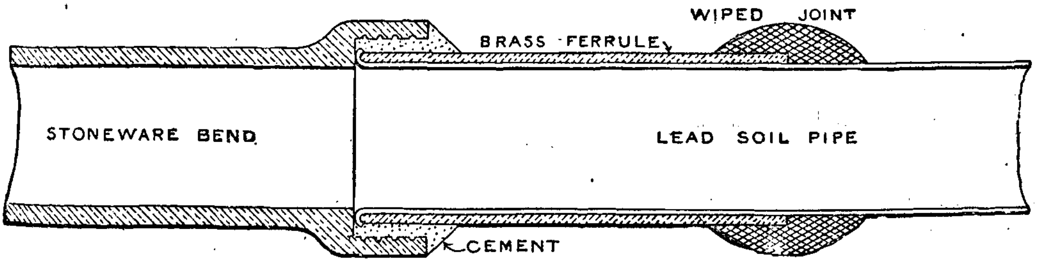

WIPED JOINT
BRASS FERRULE
STONEWARE BEND
LEAD SOIL PIPE
CEMENT

Fig. 1485.—Stoneware Bend Jointed to Foot of Lead Soil Pipe.

say 1 in 100 or 1 in 120. The gradient is kept uniform by testing it with boning rods. The pipes, after being thoroughly examined for defects, are laid with the sockets pointing upstream, beginning with the lower end. A small sinking is made under each socket, so that the pipe shall rest upon its body, and to give room for making the joint. The joint is made by taking a strand of spun yarn, dipping it into cement grout, and winding loosely round the end of the spigot; then the spigot is inserted, and the yarn (or gaskin) stemmed home, and the joint filled with neat Portland cement mortar, or 1 cement to 1 sand, and worked off with the trowel to an angle of 45°. The inside of the pipe is then cleared with a half-round wooden rake (called half-round, but really not more than one-third of a circle), or with a piece of sacking tied on the end of a stick; but good workmen will not let any cement get through to the inside. After completion, and the satisfactory application of the water test with not less than 2 ft. head of water at the top of the drain, the trench may be filled in, very carefully at first, so as not to disturb the pipes, and packing as solidly as possible under their sides. The remainder should be filled in in layers, ramming chiefly at the two sides until the earth is well consolidated. The earth may need watering, and a second ramming, before relaying the paving, which should then be bedded on sand or mortar, any broken flags being discarded for new ones.

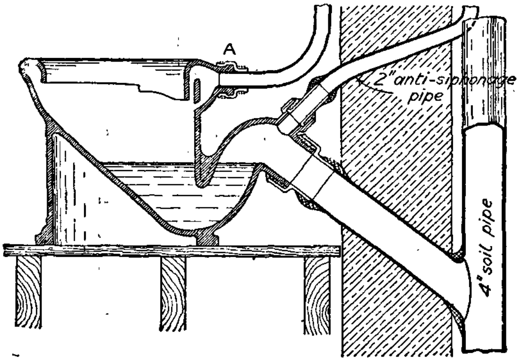


Fig. 1486.—Section of Wash-Down Water Closet.

French Drain.

A French drain may be described as a horizontal trench or an inclined trench, or a vertical cavity; these excavations are filled with rubble, which is generally chalk. The horizontal trench may be constructed on rising ground on the higher side of a house, and covered with brushwood and earth, in order to cut off the surface water from the building. The inclined trench may be constructed (at distances apart of half a chain) on the sides of a railway cutting (in clay), in order to minimise the risk of slips. The vertical cavity is constructed at the back of a retaining wall in order to prevent the accumulation of water and to lead it to the weep holes. The trench when laid at the back of a house should as a rule be as deep as the bottom of the foundations; but if the fall of the ground is so great that the foundations are stepped, the trench cannot generally be made deeper than the nearest foundations. The distance of the trench from the house will depend on circumstances; but the surface of the ground should be level between the drain and the house, so that the distance will probably be limited to 6 ft. or 8 ft. The depth of rubble should in an ordinary case be from 12 in. to 18 in.

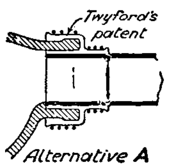


Fig. 1487.—Connection of Pan and Soil Pipe.

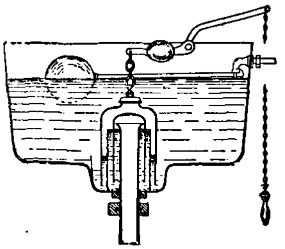


Fig. 1488.—Section of Syer's Patent Flushing Tank.

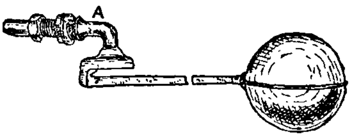


Fig. 1489.—Ball Valve for Waste Preventer.

Water Closet Construction.

Figs. 1480 to 1485 refer to a water closet on an upper floor. Fig. 1480 is a section across the seat, front to back, and shows a pedestal washdown pan, waste water preventer, and all necessary pipes and traps from the main house drain upwards. Fig. 1481 shows the stoneware bend which connects the soil pipe with the drain; Fig. 1482, the top end of the soil pipe, forming the ventilator and covered by a wire cage to prevent birds from getting inside the pipe. All the above illustrations are drawn $\frac{1}{16}$ full size. The enlarged drawings show clearly all joints between the different parts. Fig. 1483 shows an enlarged section of the joint A which connects the pan rim with the flushing pipe; Fig. 1484, an enlarged section of the joint B which connects the pan and the soil pipe; Fig. 1485, an enlarged section of the joint C which connects the soil pipe with the stoneware bend. When a w.c. on a floor above is connected to the same soil pipe, an anti-siphonage pipe must be added as shown by dotted lines in Fig. 1480. Fig. 1486 shows a section of a wash-down water closet with the connection of flush pipe to basin by indiarubber cone and copper wire, the trap and connection to lead soil pipe with brass ferrule and wiped joint, and the anti-siphonage pipe connected in the same way. The joint A can be made as in Fig. 1487 if desired. The flushing tank (Syer's patent) is shown in section by Fig. 1488. This is fixed 5 ft. above the seat with a 1¼-in. flushing pipe. When the handle is pulled, the outer bell is raised, and the water in the lower part comes up with it until the level reaches the top of the bell-mouth flushing pipe, when the water runs over and causes an induced current, which rapidly fills the pipe, and siphonic action ensues. The ball valve of waste preventer is shown by Fig. 1489, an enlarged section of the actual valve part being presented by Fig. 1490. When the ball rises the valve in the supply pipe A shuts, and when it falls the valve opens.

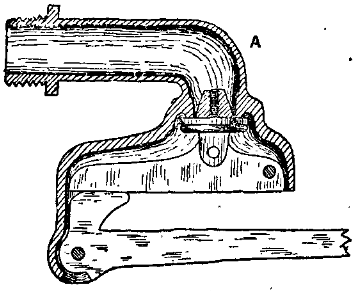


Fig. 1490.—Section of Valve.

Intercepting Traps.

Fig. 1491 shows a D-trap, an obsolete form which is apt to get very foul in use owing to the number of sharp angles, and is also particularly liable to corrosion and damage. Fig. 1492 shows an S-trap, having the minimum obstruction for a given depth of seal, sometimes made with a cleaning-plug in the bottom. Fig. 1493 shows a P-trap, similar to the last, but having the outgo horizontal or inclined, instead of vertical, to suit a waste pipe going through a wall.

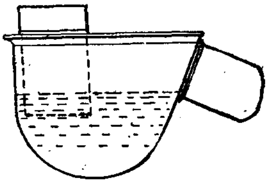


Fig. 1491.—D-Trap (Obsolete).

Siphoning in W.C. Trap.

Siphoning is liable to be produced in the trap of a w.c. when it has no anti-siphonage pipe and there is another w.c. on a higher floor; also with a single w.c. when the soil pipe is not carried up as a ventilator. The rush of water down the soil pipe past the junction causes a partial vacuum, and the pressure of air in the basin forces some of the water over and reduces the seal. It may in extreme cases leave the trap unsealed, and hence all sanitary authorities insist upon a 2-in. anti-siphonage pipe leading from the top of the bend into the ventilating pipe which terminates above the roof, so that the air in it will supersede as fast

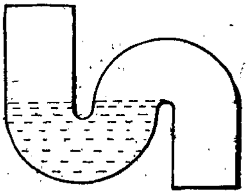


Fig. 1492.—S-Trap.

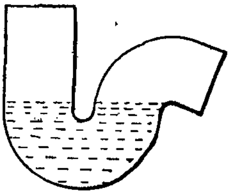


Fig. 1493.—P-Trap.

as may be needed the air drawn away. The anti-siphonage pipe is shown by dotted lines in Fig. 1480, and in full lines by Fig. 1486.

Connecting Rainwater Pipes.

If rainwater pipes are connected direct with the soil pipes, there is a possibility of sewer gas finding its way up the rainwater pipes and under the eaves of the roof into the dwelling, or through open joints in the pipe into any adjacent windows. It is generally considered that the best method is for them to discharge in the open air over a short channel leading to a trapped gully connected to the soil pipe, so that, even if the water seal dries up or is blown through, the gas would not enter the pipe ; but it is very doubtful whether, in the event of the seal blowing, it would not be better to have the rainwater pipe directly over the trap, so as to catch any accidental escape of sewer gas rather than to let it discharge into the open air at ground level.

Healthy and Unhealthy Construction.

The essential points of healthy construction are: (*a*) A site which is clean and dry; (*b*) efficient protection from the admission of air and moisture from the subsoil into the house; (*c*) walls and roofs which will effectually keep out wet, and to some extent be proof against fluctuations of heat and cold; (*d*) materials sound in quality and free from organic impurities; (*e*) rooms of sufficient size for habitation, properly lighted, and provided with suitable means of ventilation. The principal sources of unhealthiness in dwellings are: Building on made ground ; wet subsoil; damp walls; rotten floors; dead vermin; drains untrapped or leaking; ventilating shafts improperly placed; poisonous wall-papers; non-removal of old wall-paper; leakage of gas; broken slates and defective gutters; low ceilings and small windows; fireplace openings and flues blocked up ; polluted water supply; foul cisterns; non-removal of house refuse. The chief enactments which give control over these matters are: The Public Health Acts, 1875 and 1891; The Housing of the Working Classes Act, 1890; The London Building Act, 1894; and various bye-laws of the County and District Councils. Unhealthy situations are, generally speaking, those which are low-lying especially if on marshy ground or surrounded by large trees. Some situations are rendered unhealthy, although not naturally so, by the construction of cemeteries, hospitals, refuse destructors, brickyards, chemical works, sewage farms, etc. Unhealthy soils are clay, peat, and made earth, especially if the ground-water level is near the surface. Some are only indirectly unhealthy, like chalk, which makes the water "hard." Sand and gravel are generally good, but if subsoil water is within 4 ft. of the surface the site will be unhealthy. Precautions to be adopted are: Build on high ground, and if not on top of hill, direct water from higher ground away from site; drain off subsoil water; remove large trees from immediate vicinity; lay 6 in. of cement concrete over site; build damp-proof course in walls; protect walls exposed to driving rain or sea spray by tarring, covering with slates on battens, or hanging tiles, or facing with glazed bricks pointed in cement, or rendering in Portland cement, or constructing the walls with an outer skin and an air space. Provide air bricks for ventilating floors.

WARMING, VENTILATING, ELECTRIC WIRING, ETC.

Hot-Water Cylinder System, Low Pressure.

FIG. 1494 is a section showing the general principle of the hot-water cylinder system, low pressure. The boiler B, at the back of the kitchen range, has a stop valve S V to guard against explosions, and having 1¼-in. flow pipes F P, and return pipes R P to the hot-water cylinder C, giving a short circuit. This return pipe has a stopcock S C for shutting off at night to stop the circulation without endangering the boiler, as if left closed when the fire is made up, only cold water can be drawn. The cylinder of galvanised wrought iron holds 40 gal. and upwards, has a connection from the cold-water system C W, and a stopcock S C to shut off the supply in case of repairs to boiler, etc. In the top of the cylinder are inserted the main flow F P and return pipes R P, the latter being continued down to near the bottom of the cylinder. There are also a drain pipe and cock from bottom of cylinder for cleaning out. The secondary flow and return pipes S F P, S R P, are needed on account of the distance of the scullery sink from boiler. The supply to the butler's pantry, housemaid's sink, and bath is taken off the main return pipe to assist in maintaining the full circulation. The expansion pipe E P is fixed at the highest point of the system, and turned over the cistern to allow of the expansion of the water due to heat when none is being drawn off, and to permit of the escape of any air contained in the water.

Cylinder System applied to Three-Storey House.

Fig. 1495 shows the arrangement when a three-storey house is to be fitted with a hot-water supply, on the cylinder system, from the kitchen range. The draw-offs are as follows: Kitchen floor, scullery sink 22 ft. from the boiler. Ground floor, hot-water pipe rises through pantry, near sink; lavatory basin 18 ft. from same. First floor, hot-water pipe rises close to housemaid's sink, and bath is 9 ft. from the same.

Hot-Water System Connections.

A boot boiler, hot-water cylinder, and cold-water supply tank are to be connected, and there is to be a pipe with bib-cock from which a supply of hot water may be obtained. The arrangement will be as shown in Fig. 1496. A recent examination question specified that there should be three pipes to the boiler—namely, the cold-water and the two circulating pipes—and stated that it is a common error to bring the cold water into the cylinder every time there is a draw-off. The reason why it is necessary to have a cold-water tank connected with the main by a ball-cock is that if connected direct to the water main, the expansion pipe would overflow; if not connected at all, there would be no fresh supply to make good the loss as the cistern emptied; but, being connected by a ball-cock, the water is kept at approximately a constant level while the main is under pressure, although with an intermittent service it may happen that the cistern is emptied and no more hot water can be drawn. The writer is of opinion that taking the cold water straight to the boiler, as above described, might sometimes lead to serious results; for instance, if the system had been standing idle for some time with a good fire going and a furred-up boiler, the metal might be approaching a red heat. If, then, hot water were drawn off and cold water came suddenly into the boiler, it might crack the scale and reach the hot metal with explosive effect. The usual method is to connect the cold supply to the bottom of the cylinder, as shown in Fig. 1469.

High-Pressure System for Church.

The design of a high-pressure small tube hot-water heating system for a small church 18 ft. high is work for a specialist, as there are some thirty different systems of heating, but this was asked for by an examination question set a few years ago. The system shown by Fig. 1497 is known as Perkins's, after the name of the inventor. It is more than fifty years old, but has never been much in favour, as it is more costly than the ordinary low-pressure systems, and requires skilled supervision. The inventor used to say that he could make the water red-hot, but red-hot water would generally be considered rather a dangerous neighbour. The pipes are filled with water and hermetically sealed, so that there is no steam space, and no evaporation or waste unless a leak should occur. The expansion of the heated water is provided for by a small expansion chamber like a closed air vessel. There is a modification of this system, known as the safety, or double-safety arrangement, in which a self-acting valve or valves, placed in the water-supply cistern, provide the necessary means of expansion for the heated water. The space to be warmed is 60 ft. by 26 ft. by 18 ft. = say 28,000 cub. ft. Allow say 1 ft. run of pipe to 50 cub. ft.; 28,000 ÷ 50 = 560 ft. run. The available lengths of wall are 60 ft., 26 ft., and 60 ft. = 146 ft., so that four rows of piping would be required.

Specification.—Preliminary clauses and general conditions as usual. Follow with clauses relating to building of furnace-room and chimney shaft, with any incidental matters. The heating apparatus is to consist of special wrought-iron pipe having an internal diameter of $\frac{7}{8}$ in., and an external diameter of $1\frac{5}{16}$ in., tested by hydraulic pressure to 2,000 lb. per sq. in. before delivery, and certificates to be furnished to that effect. The boiler is to consist of not less than 80 ft. of this pipe, formed into a true coil with all necessary firebars, firebricks, furnace doors, casing, covers, flue connection to chimney, soot-doors, damper, etc. Proper stoking tools are also to be provided. The pipes from top and bottom of coil are to branch where they enter the church, and be laid throughout the church in four lines on iron hook standards fixed in recesses provided along base of walls. An expansion chamber to be fixed at the farthest point from the boiler, and a supply cistern on the opposite side. A diagrammatic perspective view of the pipes and boiler is shown in the accompanying sketch [that is, in the sketch accompanying the specification]. All cocks, valves, couplings, bends, tees, and everything necessary to make the installation complete, and to put it in working order, including the laying of the water supply,

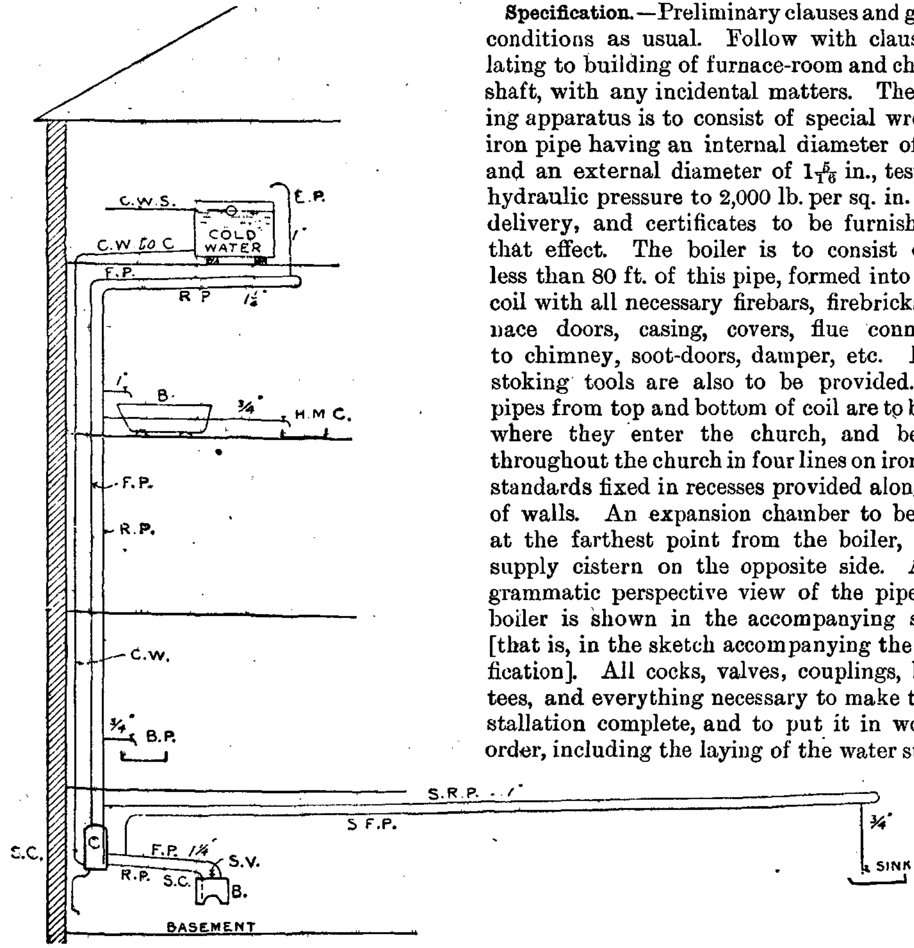


Fig. 1494.—Low Pressure Hot-Water Cylinder System, for Private House.

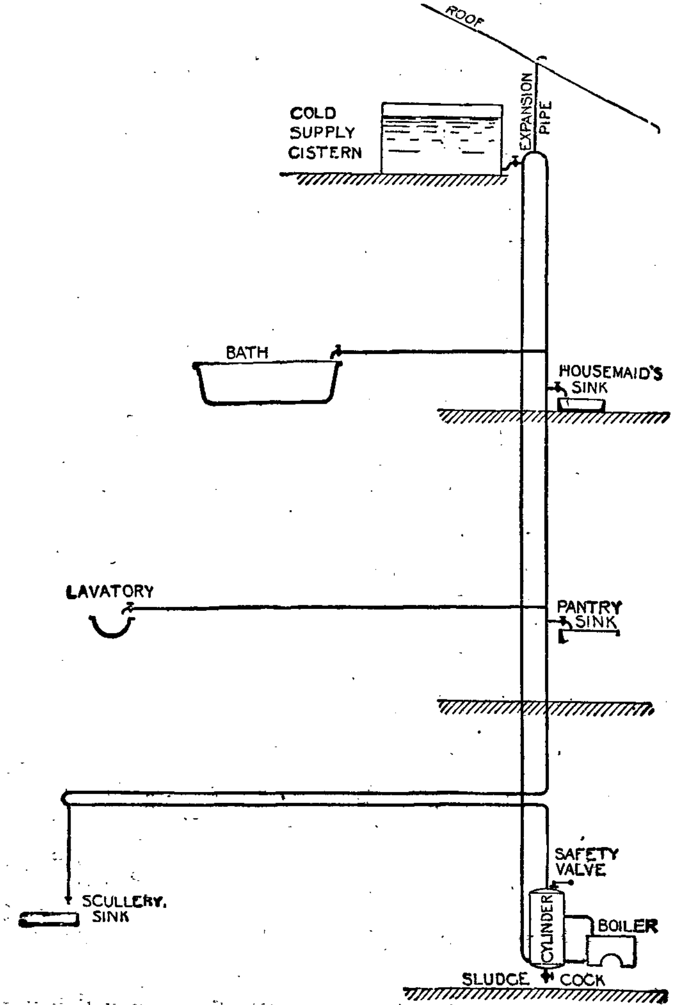

ROOF
COLD SUPPLY CISTERN
EXPANSION PIPE
BATH
HOUSEMAID'S SINK
LAVATORY
PANTRY SINK
SCULLERY SINK
SAFETY VALVE
CYLINDER
BOILER
SLUDGE COCK

Fig. 1495.—Skeleton Diagram of Hot-Water Supply of Three-Storey House, on the Cylinder System, from Kitchen Range.

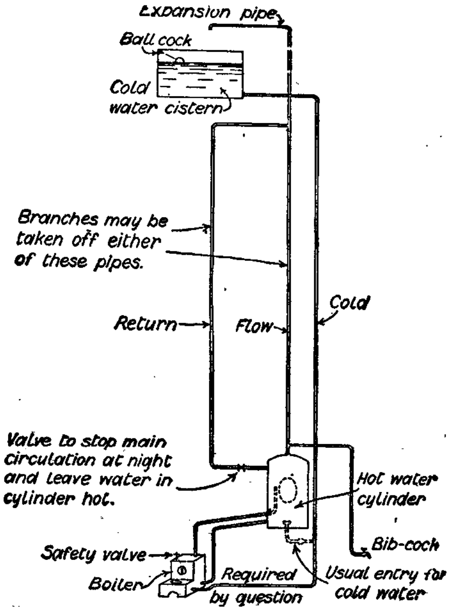

Expansion pipe
Ball cock
Cold water cistern
Branches may be taken off either of these pipes.
Return
Flow
Cold
Valve to stop main circulation at night and leave water in cylinder hot.
Hot water cylinder
Bib-cock
Safety valve
Boiler
Required by question
Usual entry for cold water

Fig. 1496.—Doubtful Hot-Water System.

are to be provided, whether specified or not. When completed, the apparatus is to be filled and warmed, and tested by hydraulic pressure to 1,000 lb. per sq. in., in the presence of the architect. If tight at all points under this pressure it is then to be heated to the usual working temperature of, say, 300° F.; and after cooling, it is again to be tested by hydraulic pressure to not less than 750 lb. per sq. in. A perforated brass casing of approved design to be supplied and fixed in front of the recess containing the pipes.

Plenum System of Ventilation.

The plenum system of ventilation consists of forcing air into a building, to take the place of air that has been fouled by respiration or combustion. It involves the use of mechanism, such as a fan driven by an electric motor, or in larger installations a Blackman air propeller driven by a steam, gas, or oil engine. The air should be purified from dust and floating germs by being passed through screens; and the additional treatment of warming in winter and cooling in summer is generally recommended. The warming may be done by passing the air, after purification, over a series of hot-water pipes or steam pipes, for the purpose of raising it to a comfortable temperature, and not for warming the building. The cooling may be done by passing the air over the same pipes converted into refrigerating pipes for the summer, or over a separate system of refrigerating pipes. The air is usually carried by wooden or metal trunks, and discharged at some height above the floor. The foul air usually obtains egress by means of trunks with openings near the floor and outlets on the roof, and also by the flues from open fireplaces where such exist. The question of plenum or injection versus vacuum or extraction ventilation is one that has been debated with vehemence, each system having uncompromising advocates. Much of the objection raised to the plenum system is probably due to the over-warming of the air taking away all its freshness, and the too great localisation of the inlets and outlets, which should be very widely distributed to produce good results. The question has been prejudiced by the introduction of the terms "artificial" and "natural" ventilation, neither of which is strictly applicable to either of the systems. The plenum method has also been further stigmatised by calling it the down-draught method. The writer's preference is for the admission of fresh air not higher than 6 ft. from floor level, and preferably lower if the inlets are well distributed, and for the emission of foul air through the ceiling, whether the plenum or the vacuum method be applied, or a combination of them. The tendency of heated air is to rise, and as the air is heated at the same time that it is fouled by either respiration or combustion, the most direct method would seem to be to inject pure air at a low level and extract foul air at a higher level.

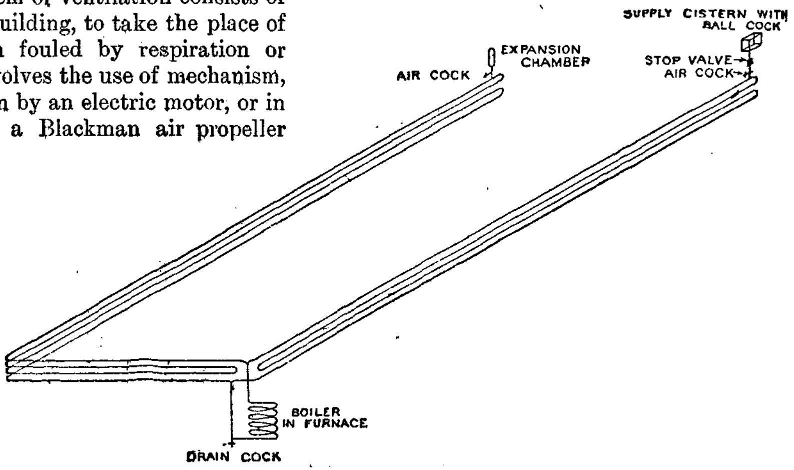


Fig. 1497.—High-Pressure Hot-Water System for Church.

Some Terms Explained.

A fireplace is the opening or recess in which the heating arrangements are fixed. A grate has no flues, except the chimney above it. A kitchener or close range may have iron flues, making it self-contained, or may be set in brickwork with flues formed by the bricklayer at the time. The boiler is a small wrought-iron or cast-iron receptacle situated at the back of the grate, through which the water passes to become heated. The cylinder is a reservoir in which hot water is stored and from which it circulates

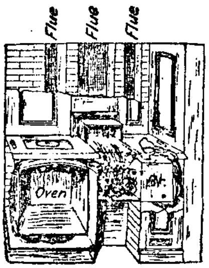

Flue
Flue
Flue
Oven

Fig. 1499.—Kitchener for Artisan's Cottage.

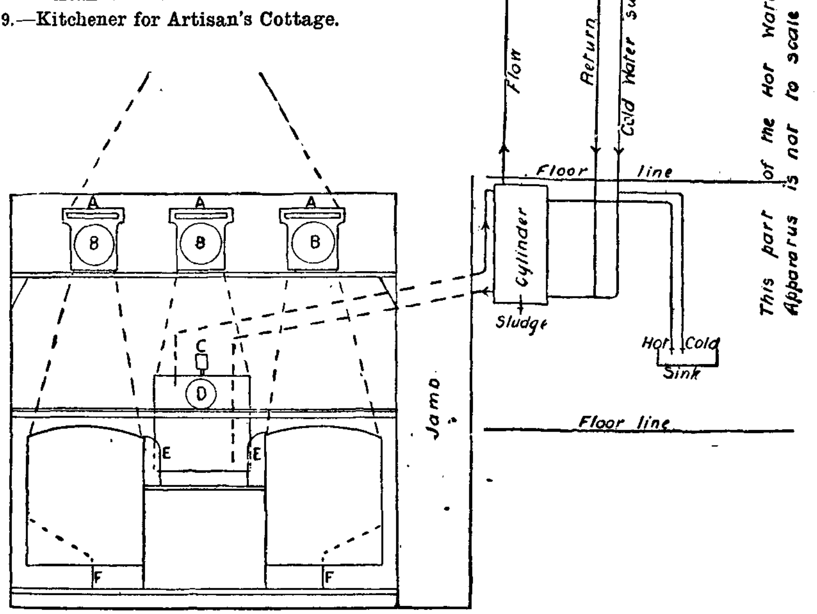

Expansion pipe
Cold Water Cistern
Flow
Cold
Hot
Floor line
Flow
Return
Cold Water supply
This part of the Hot Water Apparatus is not to scale
Floor line
Cylinder
Sludge
Hot
Cold
Sink
Floor line
Jamb
A
A
A
B
B
B
C
D
E
E
F
F

Fig. 1498.—Kitchen Range with Boot Boiler and Hot-Water System, showing Flues with Doors B B B for cleaning; Dampers A A A; Safety Valve C; Cleaning Door D; Firebricks E E; and Bafflers F F.

through the system of pipes. An expansion pipe is an extra length of pipe to provide for the expansion of water when heated; it is generally carried up from the highest point of the flow pipe and made to deliver over the cold-water cistern. Primary flow and return pipes are those which run between the boiler and cylinder, forming a short circuit for the water. Secondary flow and return pipes are those which run off from, and are brought back to, the cylinder, and it is usual to take all branch pipes from one of these. Cold-water supply is the pipe taken from the cold-water cistern to deliver either in the bottom or low down in the side of the cylinder.

Setting a Kitchen Range.

Fig. 1498 shows how to set a common type of range. The diagram makes clear the connections between the boot boiler and the hot-water cocks in scullery, bath-room, etc.; A A A indicating dampers, B B B doors of flues for cleaning, C safety valve, D cleaning door, E E firebricks, and F F bafflers. A safety valve is needed because the pipes may get furred up and stopped, or the water in them may get frozen. The valve should be placed on top of the boiler, but should not be in the flue.

Setting a Kitchener or Close Range.

Fig. 1499 shows a kitchener with oven and boiler suitable for an artisan's cottage, with the arrangement of the flues. The cleaning doors and the dampers for regulating the draught are omitted in order to expose the flues. The heat from the fire may pass direct up the middle flue, or by closing the middle damper it may be diverted to either side; and by regulating the dampers any proportion of the total heat may go in either of the three directions. To heat the oven, its damper must be opened, when the flame passes over the top down the farther side, under half the bottom, and then round the baffle plate to the other half of the bottom, and then to the side flue. If the boiler is to be heated, its damper is opened, and the flame then passes over the top, down to the far side, and under the bottom, to the flue. Iron flues save much trouble in setting, but being of wrought iron quickly rust or burn through.

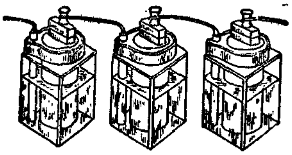


Fig. 1500.—Leclanché Battery.

Electric Bells.

Electricity is now so commonly associated with building construction that a brief mention of the working of the domestic electrical apparatus is almost indispensable. It consists of (1) the battery which generates the current; (2) the wire for the conveyance of the current; (3) the button, switch, etc., for joining up or breaking the circuit of the current; (4) the bell for use as a call or sound signal; and (5) the indicator, if required, as a visible signal. Of batteries there are several varieties, all claiming to have some special advantage, but the choice of any one is left to the architect or electrical engineer. The Leclanché battery is shown in Fig. 1500, and may consist of any number of cells from two to six. The wire generally used is copper, No. 22 s.w.g., and except where the actual connections are made this should be thoroughly insulated or covered to prevent leakage of current. A thin coating of indiarubber covered with cotton forms one of the best insulators, but the wires are often dipped in shellac, resin, or melted paraffin, all of which are fairly good insulators. For underground lines, No. 18 s.w.g. covered with guttapercha and bound with tarred tape should be used, while if the wires pass under a roadway or are in very soft ground, it is advantageous to lay them in wooden troughing or gas barrel as an extra protection. The device that is most commonly used for completing the circuit consists of a more or less ornamental box with a button in the centre, which when pressed in causes two springs to come into contact, thus completing the circuit. These springs, which are shown at A A in the interior view of the push box (Fig. 1501), are kept apart by their own tension, and should be plated with German silver. One of them should have a small projecting platinum point to insure good contact, and the whole should be kept free from dust and moisture. The bells are of two kinds, single stroke and trembling, the latter being in general use. The first, as the name implies, gives only one stroke of the hammer upon the bell, while in the second variety a continuous motion of the hammer takes place as long as the button is pressed. Another variety of the trembling bell keeps up a continuous ringing until a small arm is released, this pattern being used in connection with indicators and burglar alarms. The indicator is a rectangular box with small numbered glass windows on the front, in which corresponding white or coloured discs appear whenever any particular bell is rung. The indicator is of considerable value where a large number of bells are in use, as it would otherwise be impossible to know from which room the summons came. Another form is the pendulum indicator, in which the current causes a pendulum to swing for a given time in place of the appearing disc.

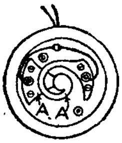


Fig. 1501.—Interior of Push Box.

Examples of Electric Bell Wiring.

An example of the arrangement of a system of electric bell wiring connected to a 6-disc indicator is shown in Fig. 1502. The wire A (technically called the copper) runs from the carbon pole of the battery G to the furthest push button. To this wire, branch wires B B from the six pushes C C are connected, and separate wires D D taken from the pushes direct to the corresponding numbers on the indicator. The indicator is shown with a relay E and local battery F, so that the main current works the indicator while the local battery works the bell H. This arrangement is more satisfactory when a trembling bell is used, as the current is intermittent, and if worked with the one battery through both bell and indicator the latter would not respond very effectively. Another arrangement of battery, bell, and indicator board for a small installation is shown in Fig. 1503. In this case there is no relay and no separate battery.

Specification for Electric Lighting of House.

This is not building construction, but work for a specialist; yet an examination question, set a few years ago, asked candidates to specify for the connection with main and the wiring and fitting of a house of about £100 rental value. In the following specification the general clauses are summarised, others being omitted to bring the answer within a reasonable compass.

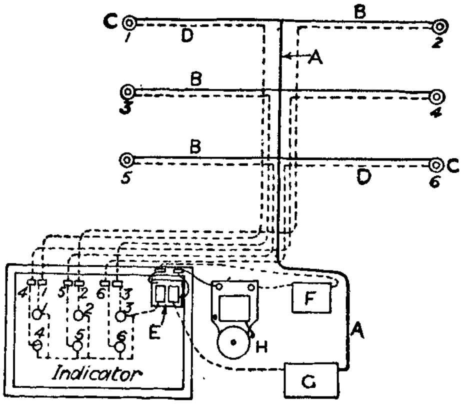


Fig. 1502.—Electric Bell Wiring to Six-disc Indicator.

Specification.

1. *Notices.*—Contractor to give all notices and pay all fees. 2. *Work comprised.*—Installation of electric light instead of gas, to include all work of every description. 3. *Plant and tools.*—Contractor to find all plant, tools, ladders, scaffolding, etc., and to remove them on completion. 4. *Wiring.*—The cables or mains to be of high-conductivity pure tinned copper wire, insulated with pure and vulcanised indiarubber and taped, the whole being vulcanised together, braided, and served and properly tested after manufacture. The E.M.F. at the distributing switchboard will be 100 volts, and there is not to be a greater drop than 2 volts in any circuit when the maximum current is being conveyed. All connections are to be made by skilled jointers, diameter of the finished joint being equal to that of the cable. The lead and return cables are to be of different colours externally. The cables are to run either on the face of the walls high up, or, where required, in chases cut to receive them. Where possible they are to be laid under the floor, or in existing pipeways. In all cases the wires are to be enclosed in American whitewood casing, or in different positions in compo pipe. Where the casing is carried through brickwork an air space is, if possible, to be left round it. The wiring is to be carried out in accordance with the general rules of the Institution of Electrical Engineers and of the Fire Insurance Companies. 5. *Switches.*—All switches are to be of the tumbler type. 6. *Fuse boxes.*—All fuse boxes are to be of approved type, and are to be cased in polished teak boxes with glass lids. Each circuit is to be distinctly labelled. 7. *Lamps.*—All lamps are to be provided by the contractor, and are to be —— or —— make. 8. *Pendants.*—All pendants are to be hung from ceiling roses firmly secured to wood blocks, which are to be firmly fixed to the ceiling joists. These ceiling roses are to be such that

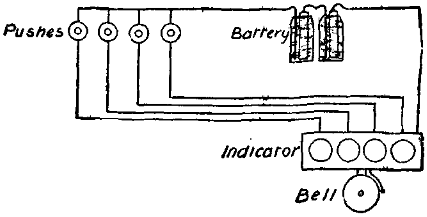


Fig. 1503.—Electric Bell Wiring to Four-disc Indicator.

the weight of the fitting is not taken by the electrical connection. 9. *Distributing switchboard.*—The current from the main is to be brought through the company's meter, and D.P. fuse-box to a D.P. switch, and a distributing switchboard placed on the wall in an approved position. The switchboard is to be of marble, mounted in a teak frame with hinged plate glass door, and provided with a lock and two keys. Care is to be taken that sufficient clearance is left between the back of the connections and the wall. All circuit switches are to be double pole quick release, and all fuses are to be double pole. The various circuits are to be labelled with ivory labels. 10. *Circuits.*—There are to be in all three circuits, as follows:—I. Basement and cellars. Kitchen. II. Ground floor and conservatory. III. First floor and second floor. Each subsidiary circuit is to be connected to a distributing fuse-box. 11. *Electric Light fittings.*—These are to be generally as indicated in the schedule, and are to be approved before being fixed. 12. *Tests.*—At the conclusion of the wiring, careful insulation tests, both between conductors and to earth, will be taken of each circuit, and the result must be to the satisfaction of the consulting engineer and of the Fire Insurance Companies. In the event of the voltage drop in any circuit being found to be more than the 2 volts specified, the wires will have to be taken out and larger wires put in. 13. *Payment.*—Payment of 80 per cent. of the contract sum will be made upon the certificate of the consulting engineer on the satisfactory completion and trial of the work. The remaining 20 per cent. will be retained until the expiration of the period of maintenance of six months.

Schedule of Lights.

This should be given in detail for each circuit, stating the position, character of lamp type of fitting, number of points, number of lights, candle power of lights, number of switches and wall plugs, etc. Summarising these, the estimate would be as follows:—

	£	*s.*	*d.*
No. 1 circuit 7 lights @ 20s. average	7	0	0
No. 2 „ 10 „ „ „	10	0	0
No. 3 „ 10 „ „ „	10	0	0
Extra for pendants, 3 electroliers, 7 fancy brackets, and main switchboard	35	0	0
	62	0	0

Say £65.

Add engineers' fees, expenses, etc.

PAINT AND PAINTING.

Ordinary Oil Paint.

THE commercial forms of the materials for ordinary lead paint are white-lead in small casks, linseed oil in drums, patent driers in tins, powder colours in casks. The white-lead, if dry, is broken up and ground on a stone muller to a uniform paste with raw linseed oil, but it is often sold ready mixed to a paste by being ground in the oil after manufacture. A sufficient quantity of this paste is then worked up with a palette knife and more oil and put into a paint-pot. The colour is then added and the paint worked up with more oil and turps. It is then strained, if necessary, by working it through canvas to clear it from lumps, skin, dirt, etc., and should now be of uniform consistency like a thick cream. Before use it is thinned with oil and turps called thinnings, and driers are also added. For outside work, half the oil to be boiled oil, and only one-third quantity of turps to be used. The preservative action of paint depends upon the linseed oil or other vehicle soaking into the pores of the material painted, and drying into a resinous compound which keeps out the air and prevents decay. The drying of paint is due to the evaporation of the volatile portions of the ingredients, as turps and moisture, and to the oxidation of the linseed oil by the atmosphere and by special ingredients rich in oxygen added for the purpose, such as litharge for ordinary paint, and white vitriol for light tints. The drier mixed with the paint causes the oil to solidify and harden more quickly.

Materials Used in Making Paint.

Boiled Oil would be used in preference to raw oil, or in equal quantities with it, for all outside work. When the work is exposed to the sun, more turps is added to prevent blistering.

Dutch White is a mixture of 1 part of white-lead to 3 parts of sulphate of baryta.

Raw (Linseed) Oil is obtained by allowing the oil, as first expressed from the seed, to settle until it can be drawn off clear. Boiled oil is composed of linseed oil mixed with certain driers and raised to a high temperature. Raw oil is suitable for internal work, for delicate tints, and for grinding up colours. Boiled oil is used for outside work, and in all cases in which rapidity in drying is required. Their drying qualities would be tested by spreading a film upon glass, or other smooth non-absorbent material, which in the case of raw oil would take from two to three days to dry, and in the case of boiled oil from twelve to twenty-four hours, according to the state of the weather.

Venice White is a mixture of 1 part of white-lead to 1 part of sulphate of baryta.

White-lead is obtained pure by placing gratings of pure lead in tan and exposing them to the fumes of acetic acid; by this they are corroded, and covered with a crust of carbonate, which is removed and ground to a fine powder. Its drawbacks are that of blackening when exposed to sulphur acids, and of being injurious to those who handle it. The following materials are used as a substitute for white-lead: Zinc white, or oxide of zinc, suitable for use where there are vapours containing sulphur compounds, also where air is contaminated with decaying animal matter. Baryta white, or sulphate of baryta, for similar purposes.

Zinc White would be used in preference to white-lead for any inside work exposed to the fumes of sulphur compounds—for example, a chemical laboratory—and where the smell of lead paint is objected to. It does not weather well for outside work unless the purest white oxide is used.

Preparing a Pot of Paint.

Sufficient white-lead is taken from the cask and put in a paint-pot with linseed oil, and mixed up with a stick, and the other ingredients are then added—say, in all, 13 lb. of white-lead, 2½ pints raw linseed oil, 1½ pints turpentine, ¼ lb. driers to each 100 square yards. A small quantity of colour (say burnt umber) is then added to make the required shade of the finishing coat when "stone colour" is required. A second paint-pot, clean inside, is then taken and covered with canvas tied on tightly, and the mixed paint worked through with a brush to remove all pieces of skin and hair. The cover being taken off, the paint is then ready for use. If necessary, a little more turps may be added to cause the paint to dry flat, assuming that this is the final coat.

Painting a New Inside Door.

The door, having been glass-papered from the bench and dusted down, has the knots killed by a solution of shellac in methylated spirits being painted on them. The priming coat, consisting of ½ lb. red-lead, 8 lb. white-lead, 2 pints boiled linseed oil, 1 pint raw linseed oil, and 1½ oz. driers, is then mixed, and the door painted all over with it. When dry and hard the door is glass-papered again and dusted, and all holes and cracks stopped with glaziers' putty. The second coat, consisting of 7½ lb. of white-lead, 1 pint boiled linseed oil, ¾ pint raw linseed oil, ¾ pint turps, and 2 oz. driers, with enough red-lead to tinge it flesh colour, is then applied, and when dry the door is again rubbed down and dusted ready for the third coat. This would be composed of 6½ lb. white-lead, 1 pint boiled linseed oil, ½ pint raw linseed oil, ¾ pint turps, 2 oz. driers, and sufficient colour (say raw umber) to approximate to the finished tint. The final coat would be composed of 6½ lb. white-lead, ½ pint boiled linseed oil, ¼ pint raw linseed oil, 1¼ pint turps, 2 oz. driers, and enough colour to make it of the tint required. The quantities named would make enough paint for half a dozen doors and their architraves and linings. It is not possible to paint satisfactorily with fewer than two coats after the priming has been laid on, even when the paint is mixed thick. The same remark applies to renovating old work. A dull matt surface is produced by using turps.

Painting a New Outside Door.

Preparation of Woodwork.—The woodwork should be thoroughly seasoned and dry before being painted. The surface should be planed clean and swept free from dust. All nails should be punched in so that their heads are below the surface.

Killing Knots.—The surface is "knotted" before the priming coat is put on; that is, the knots are "killed" or covered with a substance through which the resin cannot exude, as red lead and size, shellac dissolved in naphtha, or silver leaf (tinfoil). The knots may be killed in either of the following ways:

Ordinary or Size Knotting is applied in two coats. The first coat is made by grinding red-lead in water and mixing it with strong glue size. It is used hot, and dries in about ten minutes. Second coat consists of red-lead ground in oil and thinned with boiled oil and turpentine.

Patent Knotting is chiefly shellac dissolved in naphtha. A recipe for patent knotting is as follows: Add together ¼ pint japanner's gold size, 1 teaspoonful red-lead, 1 pint of wood naphtha, 7 oz. orange shellac, the mixture to be kept in a warm place whilst the shellac is dissolving, and to be shaken frequently.

Lime Knotting.—The knot may be covered with hot lime, which, after being left on for about twenty-four hours, may then be scraped off and the surface coated with size knotting; if this does not kill them they may be coated with red and white-lead in linseed oil, and when quite dry rubbed smooth with pumice-stone, or after the application of the lime they may be ironed with a hot iron and then painted smooth. In superior work the knots may be cut out to slight depth, and the holes filled with putty made of white-lead, japan, and turpentine.

Priming Coat. — After the knotting, the priming coat is laid on, which should be of the proportions following: White-lead 10 lb., raw linseed oil 4 pints, red-lead 1 oz., litharge or patent driers 2 oz.

Stopping.—The surface should then be rubbed down with fine sandpaper or pumice-stone. and all holes stopped with putty.

Second Coat.—When the priming is dry the second coat is then applied and allowed to harden. The paint should be of the ingredients and proportions as follows: White-lead 10 lb., raw linseed oil 2½ pints, litharge or patent driers 2 oz., turps 1½ pints.

Third and Fourth Coats.—The third coat is then applied, and when it is dry and well rubbed down the finishing coat is added. The last two coats should consist of the ingredients and proportions following: White-lead 10 lb., raw linseed oil 2 pints, litharge or patent driers 2 oz., turps 2 pints. The last two coats should also contain the final colouring added in proportion to the depth of tint required. From 1 oz. to 2 oz. of colouring matter is added to every ten yards of surface to be painted, and the quantity of white-lead is reduced in proportion.

Outside Work.—If the paint is to be exposed to the sun, boiled oil should be used, and the quantity of turps in the second coat should be reduced to about one-half of that mentioned for inside work, and there should be no turps used in the remaining coats except in winter.

Graining.—Woodwork is grained to make a cheap and easily worked wood imitate one of better and more expensive quality. The ordinary priming coat is first put on as already described, whatever the final graining may be, and the holes stopped with putty. For oak graining, four or five coats of paint having been applied, the last is composed of equal parts of oil and turps, and should approximate in tint to the final colour required, after which thin glazings of terra de sienna, umber, vandyke brown, or other required tints are applied. These tints may for ordinary work be ground in water and mixed with small beer, but for oak the colour is mixed with turps and a little turpentine varnish, and its surface before it is dry is scratched over (with combs or with flat brushes dipped in turps) to imitate the grain of different woods.

Painting and Graining and Finishing a Moulded Door.

The door (internal work), being cleaned off to dimensions or tried up in place, is well rubbed down with fine glass-paper and dusted with a bench brush. The knots are then "killed" or covered with red-lead, glue and size, patent knotting, or tinfoil, to prevent the exudation of turpentine or resin from discolouring the finished work. When dry the knots are smoothed with pumice-stone and the priming coat is laid on. This is the first coat of oil colour, and is composed of ½ lb. red-lead, 16 lb. white-lead, 6 pints raw linseed oil, ¼ lb. driers for 100 yds. super.; it closes the pores of the wood and prevents the oil from the putty, and the after coats of paint, from being absorbed by the wood. When dry, the holes are stopped with glaziers' putty, and the whole surface is pumiced over and dusted. (In common work, the priming coat, called "sheep-skin," consists of red-lead or red ochre, water, and size.) The second coat consists of 15 lb. white-lead, 3½ pints raw linseed oil, 1½ pints turpentine, ¼ lb. driers per 100 yds. super. When this is dry and hard, it is rubbed down with pumice, and the third and fourth coats are put on in a similar manner; each consists of 13 lb. white-lead, 2½ pints raw linseed oil, 1½ pints turpentine, ¼ lb. driers per 100 yds. super., with sufficient ochre or umber to give a light stone colour. The graining coat is then put on, consisting of burnt umber, raw oil, and driers mixed thin, and while still wet is "combed" or scratched with steel, horn, or wood combs to imitate the grain of oak, the "flower" being produced by pieces of rag, sponge, etc. In best work another coat is laid on, called "over-graining," consisting of raw umber mixed with beer, enabling a more complete imitation. The work is then finished with two coats of best hard copal varnish. The above routine is technically described as "Clean, rub down, knot, prime, stop, and paint three oils, grain wainscot, overgrain, and twice varnish with best copal."

Painting a New Entrance Door.

To commence the description with the priming, it is assumed that the door has been cleaned off to dimensions and tried up in place, and well rubbed down and dusted, and that the knots have been properly killed. The priming coat is then laid on, composed of white-lead and a little red-lead, mixed with raw linseed oil, and a little litharge ground very fine in turpentine as a drier. After the priming coat to stop suction, there comes the stopping or filling up of the nail-holes, etc., with glaziers' putty, or, better, with hard stopping of white-lead mixed with gold size, and then the door may be hung. The work must then be rubbed smooth again with glass-paper or pumice-stone, well dusted, and the second coat put on, consisting of white-lead with a little litharge mixed with boiled linseed oil, and very little, if any, turps, to avoid blistering and sun cracks. A small portion of burnt umber should be added to give the desired colour. The third coat should consist of the same ingredients, except that if the door were not required to be varnished there should be no turps. The previous coat being dry, the graining coat is then to be applied, consisting first of an even ground of light tint composed of white-lead stained with orange chrome, linseed oil, and a little driers ; and as soon as this is dry, a graining colour of burnt umber and linseed oil with a little driers is put on, combed for the grain, and worked with pieces of rag or sponge for the flower. An extra coat of glazing or overgraining may be made of a thin wash of raw umber mixed with small beer and applied with a flat brush drawn over the surface with a wavy motion of the hand. When dry, the door should be varnished with one or two coats of best hard copal body varnish. Care must be taken that none of the painting or varnishing is done while the surface is damp, as in the early morning, or during or immediately after rain, when blistering is sure to take place.

Estimating the Cost of Painters' Work.

For ordinary work, say :

Knot, prime, and stop ...	3d.	per yard super.
First coat oil painting ...	2½d.	"
Second " ...	2d.	"
Third " ...	1½d.	"
Total ...	9d.	"

360 yds. @ 9d. = £13 10s.

For best work, say :

	£	s.	d.
KNOTTING—			
Labour and materials—			
360 yds. @ ½d. per yard	0	15	0
PRIMING—			
2 lb. red-lead @ 3¼d. per lb. ...	0	0	6
56 lb. white-lead @ 3¼d. per lb. ...	0	15	2
20 pts. raw linseed oil @ 4d. per pt.	0	6	8
1 lb. driers @ 2d.	0	0	2
Labour, 360 yds. @ 2d. per yard ...	3	0	0
STOPPING—			
7¼ lb. white-lead @ 3¼d. per lb. ...	0	2	0
18 lb. putty @ 3d. per lb.	0	4	6
Labour, 360 yds. @ 1d. per yard ...	1	10	0
PAINTING, THREE COATS—			
148 lb. white-lead @ 3¼d. per lb.	2	0	1
30 pts. raw linseed oil @ 4d. per pt.	0	10	0
20 pts. turps @ 5½d. per pint ...	0	9	2
¾ lb. driers @ 2d. per lb.	0	0	2
Labour, 3/360 yds. @ 1½d. per yard.	6	15	0
	16	8	5

Say, 1s. per yard.

These prices include 15 per cent. profit.

Whitewashing, Limewhiting, and Distempering.

Whitewashing may consist of painting over surfaces with a mixture of pure freshly slaked chalk lime in water, put on while hot, but this is perhaps more correctly known as limewhiting, and is used for sanitary reasons. Whitewashing may also consist of a mixture of whiting (pure white chalk powdered and made up again), size and water, but this is more correctly whitening, or white distemper, and is specially used for ceilings. The size should have a disinfectant added to prevent decomposition and foul smell. Alum, tallow, sulphate of zinc, skim milk and common salt have all been recommended as additions to prevent scaling or rubbing off. Coloured distemper is made by adding a small quantity of colouring matter, such as ochre or umber, to the whiting, and grinding in water before adding the size. Water paints are a variety of distemper, but it is claimed that they are free from whiting and size. They are probably silicate distempers. Col. Seddon says, "For outside walls a colouring coat of whiting and size mixed with boiled oil is often used as a cheap substitute for ordinary paint."

STRESS DIAGRAMS FOR ROOFS.

Reciprocal Diagrams.

The simplest illustration of reciprocal, or complementary, or inverse diagrams is obtained by taking three forces in equilibrium in the same plane acting on a point, as shown in Fig. 1504 by the three lines forming what is called the force diagram, or frame diagram. The spaces between the lines are numbered, and any force is named by the figures in the spaces on each side, namely, the larger force marked 1 and 3 on each side would be known as 1—3, the others being known respectively as 1—2 and 2—3. The reciprocal diagram of this is obtained by starting at point 1 (Fig. 1505), drawing line 1—3 parallel to 1—3 in Fig. 1504 and of a length equal to the magnitude of the force to any given scale, then drawing the other lines, 1—2 and 2—3 respectively, parallel to the similar force lines in Fig. 1504. This will fix their length or magnitude, which may have been previously unknown, and Fig. 1505 is then the reciprocal or opposite of Fig. 1504. It will be observed that lines meeting in a point in the frame diagram form a closed figure in the reciprocal diagram, and vice versâ. Now for the roof truss shown in the frame diagram (Fig. 1506), add the external force lines, and number all the spaces, both external and internal, in regular order. Then commence the reciprocal diagram (Fig. 1507) by drawing to scale the external forces 1 to 9, called the load line or line of loads. Draw lines in order parallel to similar lines in Fig. 1506, always working from two known points to find the third or unknown, thus 2—10, 9—10, to find point 10; 10—11, 3—11, to find point 11; 11—12, 4—12, to find point 12; 12—13, 9—13, to find point 13; and so on. When the loading is symmetrical, as in this case, it is not necessary to go beyond the half truss with the reciprocal diagram, as both halves will be similar, but it helps to check the work when the figure is completed. The fact of the figure joining up completely is generally good evidence of the accuracy of the method employed. The lengths of the lines in the reciprocal diagram measured by the same scale as the line of loads will give the stress in each member of the truss. The nature of the stress, whether tension or compression, is found by following the direction of the lines in the reciprocal diagram thus. Suppose it is desired to know the nature of the stresses in the parts meeting at the joint 3—4—

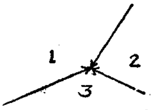


Fig. 1504.

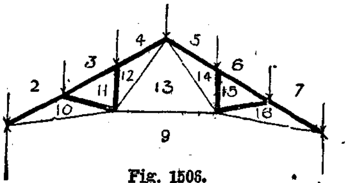


Fig. 1506.

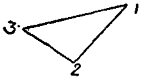


Fig. 1505.

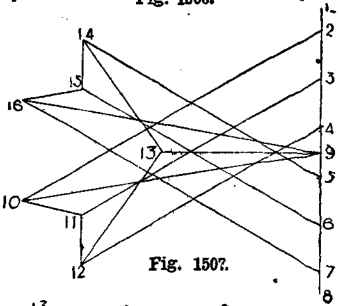


Fig. 1507.

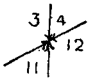


Fig. 1509.

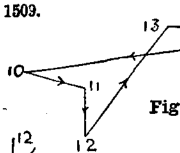


Fig. 1510.

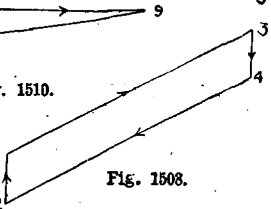


Fig. 1508.

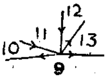


Fig. 1511.

Figs. 1504 and 1505.—Simple Reciprocal Diagrams. Figs. 1506 to 1511.—Obtaining Stress Diagrams of Roof Truss.

12—11 in Fig. 1506, look to Fig. 1507 and note the parts concerned, which are repeated for clearness in Fig. 1508. Always take the members clockwise at the joint, and start with a known direction. In this case 3—4 acts downwards, so put an arrow-head on it in Fig. 1508 and continue the arrow-heads round the figure concurrently—that is, all running the same way. Transfer the arrow-heads to that portion of the frame diagram repeated in Fig. 1509, and then it will be seen that all these bars are in compression, because the forces in them act towards the joint. Now consider the bars that meet at 11—12—13—9—10, the reciprocal diagram of which parts is repeated at Fig. 1510. In this case there is no external force to start with, but we know that 11—12 is in compression, and to indicate this in Fig. 1510 an arrow-head must be put as shown; the others must then be placed to run concurrently. Transferring these to the portion of the frame diagram repeated at Fig. 1511, it will be seen that 12—13 acts away from the joint, and therefore indicates tension; 13—9 and 9—10 act similarly away from the joint and show tension; while 10—11 is towards the joint, and therefore shows compression. With a little practice all this can be seen by inspection without making any repetitions or any marks upon the main diagrams. If arrow-heads were put upon Fig. 1507 it would be found that in certain cases they become reversed. This will be seen by comparing the arrow-head in 11—12 (Fig. 1508) with 11—12 (Fig. 1510). Reciprocal diagrams have been called the royal road to the investigation of stresses, as they simplify the work in such a marvellous manner.

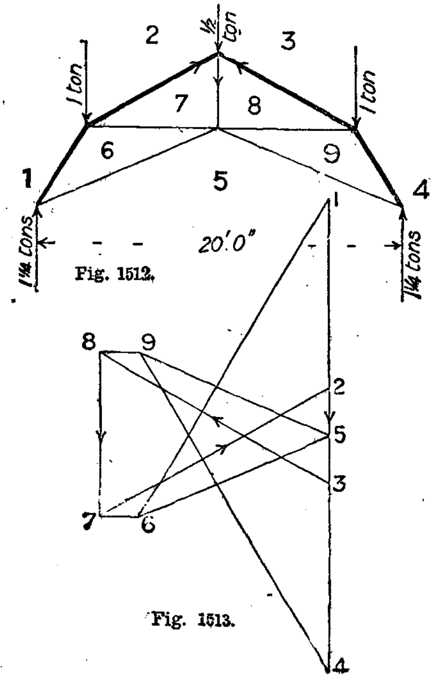


Fig. 1512.

Fig. 1513.

Figs. 1512 and 1513.—Frame and Stress Diagrams of Roof.

Bow's Notation.

Bow's notation for reciprocal diagrams consists of lettering each space in the frame diagram, whether wholly or partially enclosed; the reciprocal or stress diagram having the same letters at the ends of the lines representing the stresses in the bars. It simplifies the comparison of the two diagrams, and shows exactly, without any dimension lines, how far the measurements on the stress diagram are to be taken. Figures may with advantage be used instead of letters, without altering the principle.

Finding Parts in Tension or Compression.

As another illustration of the method of finding whether the parts are in tension or in compression, take the frame diagram as Fig. 1512, and stress diagram as Fig. 1513. The nature of the stresses is determined as follows: Take apex of truss, note that the external load acts downwards, and that the clockwise order of the figures in surrounding spaces is 2, 3, 8, 7, 2. Then refer to stress diagram, take the figures in the same order, and mark arrow-heads to show the direction. Transfer the arrow-heads to frame diagram, then those that act towards the joint show compression, and those away from the joint show tension. Any other joint may be treated similarly, working from a known to the unknown directions, with a new set of arrow-heads. Take the joint at the foot of the king-rod; there is no external force, but force 7—8 is known to be acting away from the joint because the king-rod is in tension; then, following the other figures round in the stress diagram, the tie-rods can be proved to be also in tension.

Thrust on Wall Plate.

Thrust will be caused on a wall plate only when an open span roof is adopted, or with a collar-beam roof under certain conditions. In other cases the wall plate is under a vertical load only. Let A B C (Fig. 1514) represent the section of roof; draw the "frame diagram" (Fig. 1515) with force lines to represent the load on apex, thrust on each wall plate and reaction of supports. Number the spaces separating the forces, commencing at the left and working round over the top. Now draw the loads on the "reciprocal diagram" (Fig. 1516), making 2 to 3 equal in length upon any suitable scale to the force or load on the apex which is known by the numbers of the spaces it separates—namely 2—3. Then from points 2—3 in Fig. 1516 draw lines 2—5, 3—5 parallel to the rafters 2—5 and 3—5 of Fig. 1515, and intersecting in point 5. Next from points 2 and 5 in Fig. 1516 draw lines 2—1, 5—1 parallel to 2—1, 5—1 in Fig. 1515, and then lines 3—4, 5—4 in Fig. 1516 parallel to lines 3—4, 5—4 in Fig. 1515. The stresses in the roof members will be found by scaling off the lengths of the corresponding parts of the stress diagram; for instance, the thrust in the rafters will be given by the length of 2—5 and 3—5 in Fig. 1516, because 2—3 was drawn equal to the load on apex. The horizontal outward thrust on the wall plates will be given by lines 1—2 and 3—4 of Fig. 1516, and the load to be supported by the walls will be represented by 1—5, 5—4 of the same figure. The nature of the stress in the members meeting at any point will be found as follows. Take, for instance, the apex in Fig. 1515. The three lines meeting at that point are 2—3, 3—5, 5—2, and in the reciprocal figure these three lines will form a triangle 2—3, 3—5, 5—2. Upon sides 2—3 put an arrow-head to show the direction in which the force is acting, and then add arrow-heads to the other sides of the triangle concurrent with the first—that is, running the same way round. Now compare the directions on 3—5 of Fig. 1516 with the rafter 3—5 of Fig. 1515, and it will be found that it is acting towards the apex, and therefore the rafter will be in compression. The force in 5—2 of Fig. 1516 seems to be acting the reverse way; but, comparing the position of rafter 5—2 in Fig. 1515, it will be seen that the force is acting towards the joint, making it a compressive stress. Several simple forms of roof truss should be taken, and stress diagrams drawn out to scale to obtain facility in

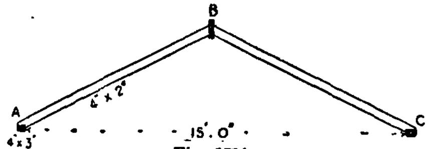


Fig. 1514.

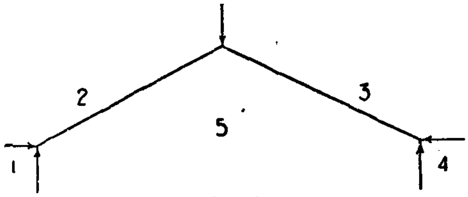


Fig. 1515.

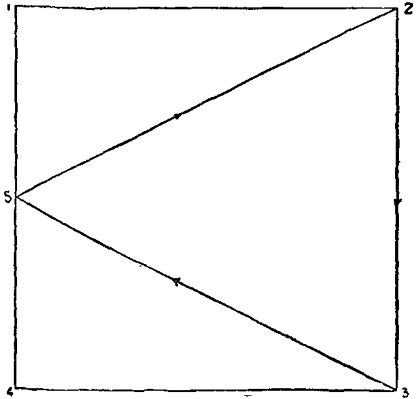
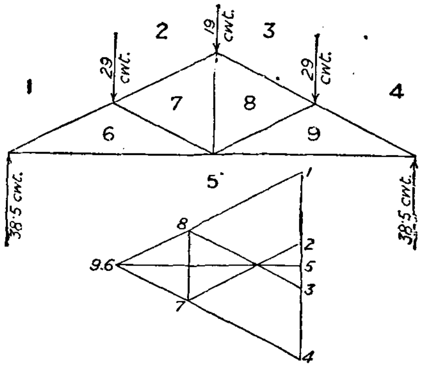


Fig. 1516

Figs. 1514 to 1516.—Obtaining Thrust on Wall Plate.



Figs. 1517 and 1517a.—Easy Case for Stress Diagram.

working more complicated cases. Figs. 1517 and 1517a show a particularly easy example given as an examination question. Another is shown by Figs. 1518 to 1520, the total distributed load over the principal rafters being given as 9·6 tons.

Iron Roof Truss, 28-ft. Span.

An iron roof consists of trusses, as shown in Fig. 1521, at 8-ft. centres. Taking the maximum load to which it would be exposed at 40 lb. per horizontal foot super., the nature and amount of the maximum stress to which each member might be exposed is shown graphically by Fig. 1522. The stress diagram is shown in Fig. 1523, the nature and amount of maximum stress being marked on each member.

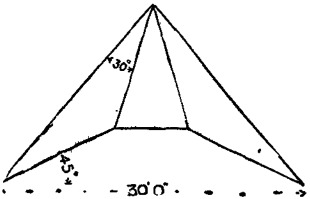


Fig. 1518.

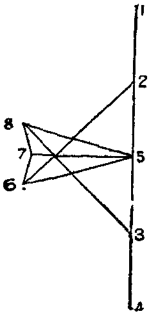


Fig. 1520.

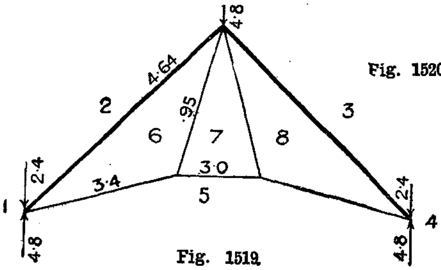


Fig. 1519.

Figs. 1518 to 1520.—Frame and Stress Diagrams.

Roof Truss with Hanging Loads.

To find graphically the stresses in the members of the roof truss shown in Fig. 1524, draw the frame diagram according to the conditions as in Fig. 1524, then proceed with the load line in the stress diagram (Fig. 1525). The first difficulty is to know the value of the reaction 6-7. This will be found by leverage of loads from the opposite end; thus $4 \times \frac{1}{4} + 3\frac{1}{2} \times \frac{2}{4} + 3 \times \frac{3}{4} + 1 \times \frac{3}{10} + 1 \times \frac{7}{10} = 1 + 1\frac{3}{4} + 2\frac{1}{4} + \cdot 3 + \cdot 7 = 6$ tons. The reaction at the other end will be the difference between 6 tons and the total load. Total load $= 4 + 3\frac{1}{2} + 3 + 1 + 1 = 12\frac{1}{2}$, $12\frac{1}{2} - 6 = 6\frac{1}{2}$ tons. After this, no further trouble will arise, and the stress diagram may be completed as shown.

Roof Truss, 45-ft. Span.

Fig. 1526 shows a Fink, French, or Belgian truss for a span of 45 ft. The stress diagram is shown in Fig. 1527, and the nature and amount of maximum stress in Fig. 1528.

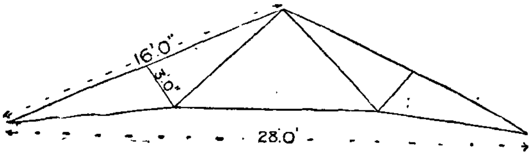


Fig. 1521.

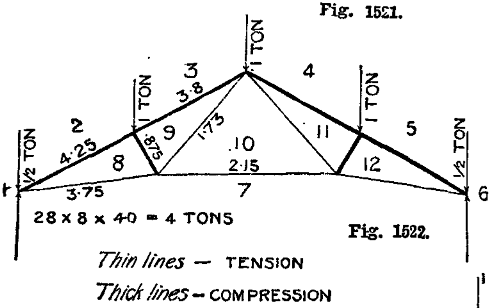


Fig. 1522.

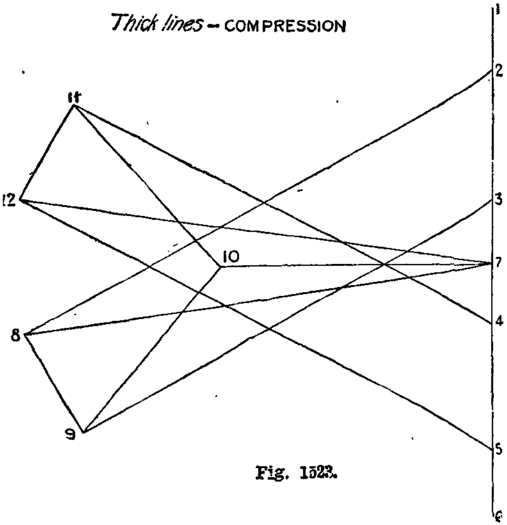


Fig. 1523.

Figs. 1521 to 1523.—Diagrams for Iron Roof Truss 28-ft. Span.

Obtaining Stress Diagram—Difficult Case.—Fig. 1529 shows a portion of a roof truss in skeleton; it is stressed by the forces shown. Fig. 1530 shows the stress diagram. The usual assumption is made that these members have their axes of symmetry in one plane, and that the lines of action of the applied forces are all in the same plane, and that all the members which meet at one point are each (if not held elsewhere) free to move round a pin at the point, and that these pins are at right angles to the plane of the forces, represented by the plane of the paper. A difficulty occurs in finding point 19 and onwards. Take 19*a*, at about the point at which it is expected that 19 should be, then work through 20*a*, 21*a*, 22*a*, and 11*a*, as if they were correct points. Now 22*a*, 19*a*, and 11*a* should be in the same straight line to give the true points for 22, 19, and 11. This may be done by trial, or easier by marking points 20*a*, 21*a*, and 22*a*, and *x* on a slip of tracing paper, and sliding it up the three parallel lines until *x* coincides with the line from 18 to 19*a*, giving the true point for 19, and the figure may then be completed in the usual way. This may also be worked by making use of substituted members, as dotted in Fig. 1531, where spaces *a b c* are substituted for 19, 20, 21, and 22. Then, constructing the stress diagram, point 11 is found, and from 18 and 11 point 19 is found, and the remainder follows without difficulty.

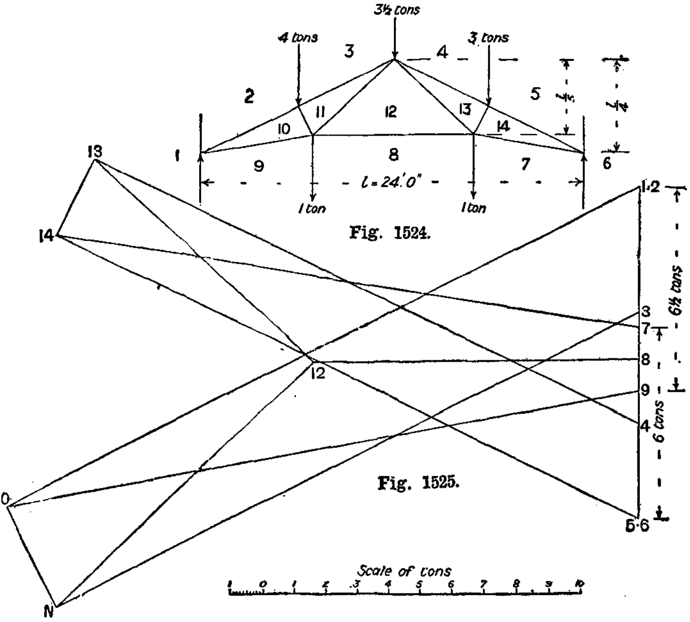
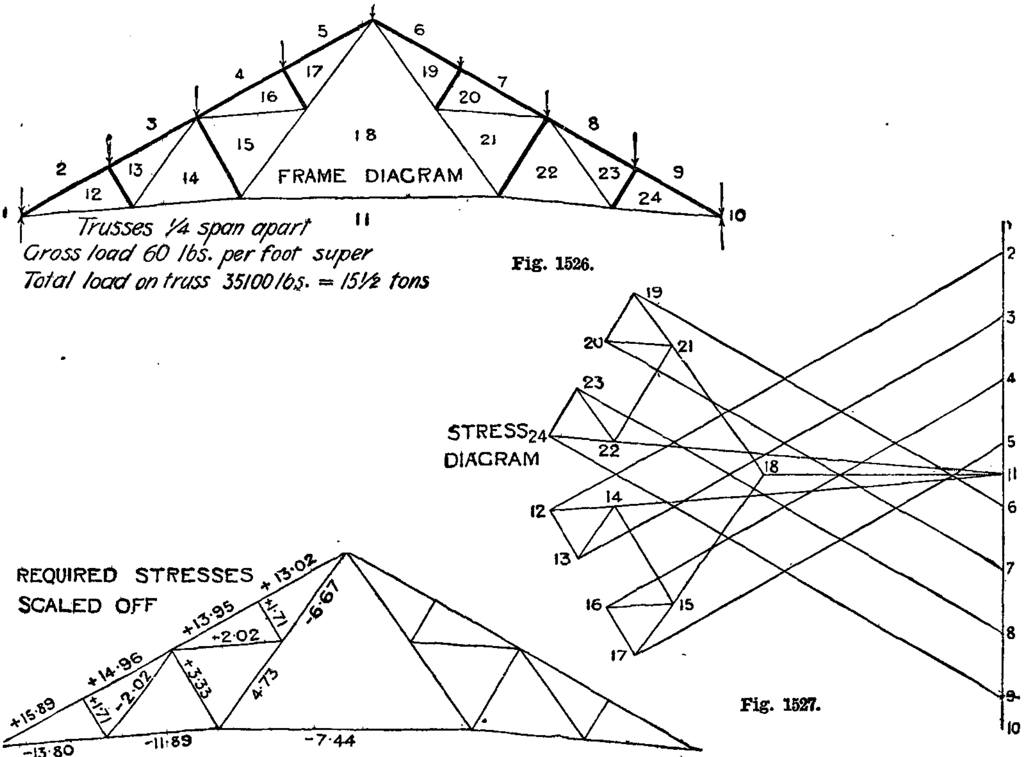


Fig. 1524.

Fig. 1525.

Figs. 1524 and 1525.—Roof Truss with Hanging Loads.

Another Case.—Assume that the roof truss shown inside a portion of Fig. 1532 carries the following loads:—(1) A dead load which amounts to 30 lb. per square foot of covered area; $\frac{2}{3}$ of this is distributed over the upper nodes and $\frac{1}{3}$ over the lower nodes. (2) A live load due to the wind, acting on the left-hand side of the truss, and equal to a normal intensity of 30 lb. per square foot of roof surface. The bars A *b*, *b a*, *a c*, *c* B, are all of equal length, and the trusses are 12 ft. apart. It is required to determine the stresses in the bars on the right-hand side of the truss due to dead load and wind. The above is an examination question, and it will be noted that the word "node" is used. The continual introduction by examiners and writers of new terms to express well-known things is to be deprecated, but has to be accepted. This is a case in point. The word "node" is borrowed from the science of acoustics, and is out of place here. A node is the point of no motion between two vibrating (or ventral) segments of a tightened string vibrating transversely. It may therefore be assumed that the word "nodes" in the question means the points of support of principal rafters, and the points marked *a b c a' b' c'* in Fig. 1532. It would have been much simpler to say, "Two-thirds of the load is distributed over the external surface of the roof and one-third over the lower ends of the struts." The frame diagram being as shown in Fig. 1533, the loads will be as marked. Then, to find the reactions, produce the lines of action of the loads to meet the main tie-rod. This is shown in Fig. 1534, and is separated from Fig. 1533 for the sake of clearness. Number the spaces as shown, and set down the load line as in Fig. 1532. Select a pole o, and from the load line draw vectors. Then, parallel to these vectors, draw the funicular polygon as shown by stroke-and-dot line in Fig. 1534. Now from o in Fig. 1532 draw a line o—18 parallel to the closing line of the funicular polygon, giving the reactions as 16—18 at the fixed end, and 18—1 for the free end. The load line for the complete stress diagram may then be set down as Fig. 1535, and the diagram worked out. A slight difficulty occurs in finding point 22. This may be done by substituting the member *a*—*b* for 19—20 and 20—21, then working as follows: 18—*a*, 4—*a*; *a*—*b*, 5—*b*; *b*—22, 13—22; which gives point 22; and the rest is worked out in the ordinary manner. The stresses in the right-hand side of the truss are then scaled off and marked on Fig. 1533 as required.



Trusses ¼ span apart
Gross load 60 lbs. per foot super
Total load on truss 35100 lbs. = 15½ tons

Fig. 1526.

Fig. 1527.

Fig. 1528.

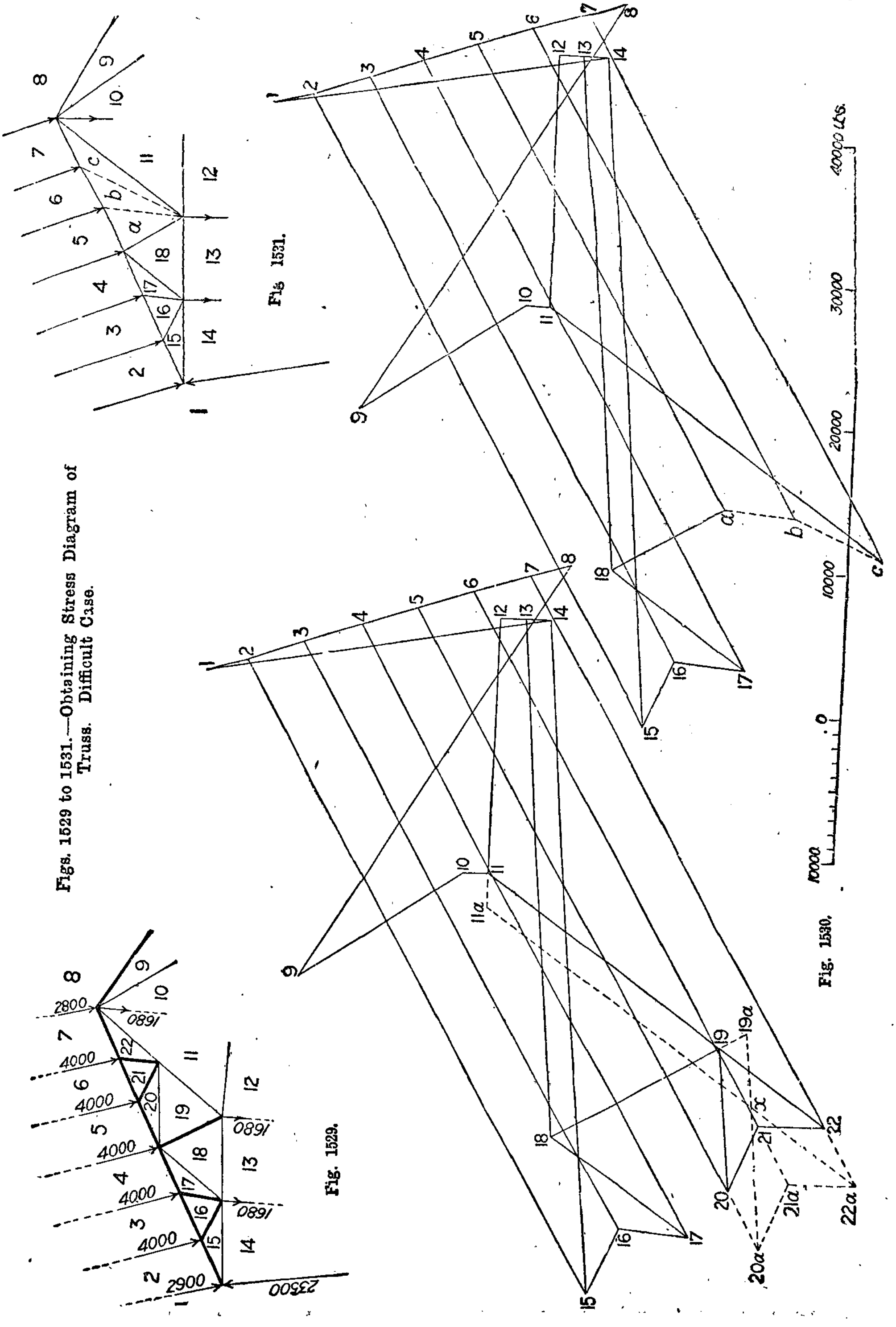
Figs. 1526 to 1528.—Diagrams for Fink Truss, Span 45 Ft.

Stresses in Large Roof Truss.

Fig. 1536 shows the outline or frame diagram of a large truss, and Fig. 1537 the corresponding stress diagram. Fig. 1538 is an enlargement of the corner of the stress diagram.

Stresses in a Roof Truss with Plate Webs.

A stress diagram is required for the truss reproduced in Fig. 1539 for dead load, and one for wind pressure. The slates are countesses, wired direct to small steel purlins. This is rather a complicated case, but it may be simplified by assuming the plate webs to be superseded by struts, and the tie girder by a rod at its centre line, as shown in the frame dia-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9
10
11
12
13
14
15
16
17
18
a
b
c
19
20
21
22
11a
19a
20a
21a
22a
x
2800
4000
4000
4000
4000
4000
2900
1680
1680
1680
23500
10000
0
10000
20000
30000
40000 lbs.

Fig. 1529.

Fig. 1530.

Fig. 1531.

Figs. 1529 to 1531.—Obtaining Stress Diagram of Truss. Difficult Case.

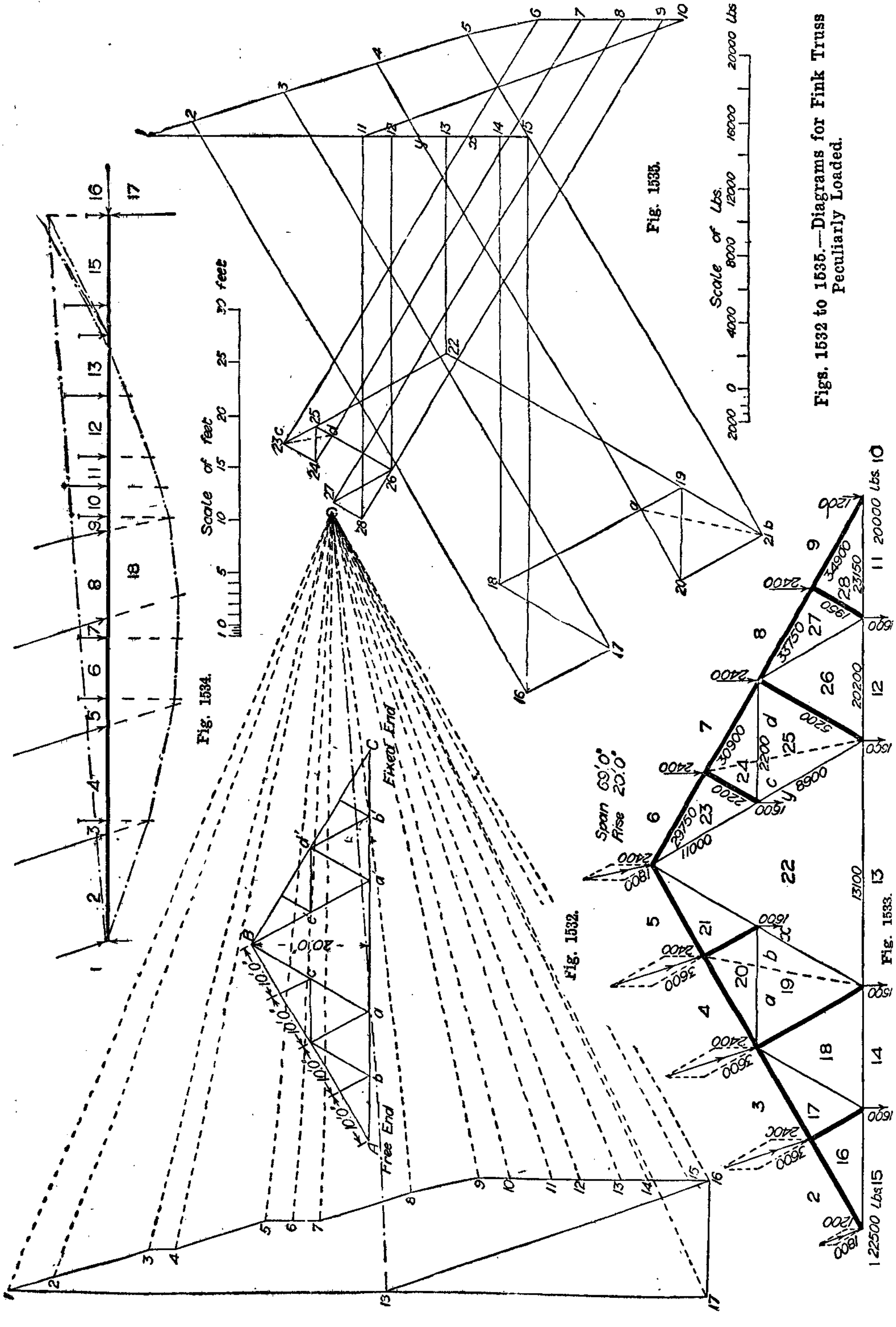
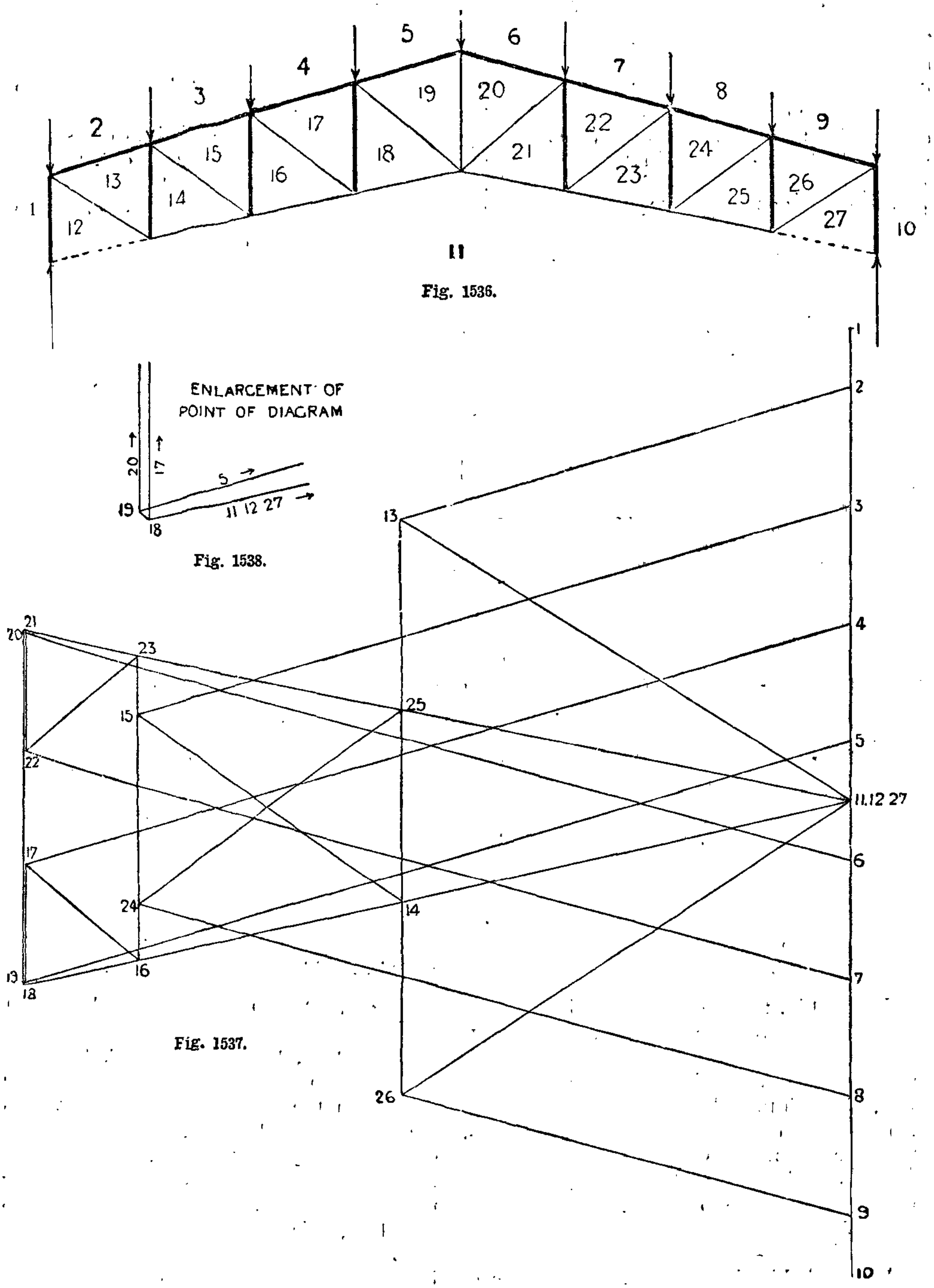


Fig. 1532.
Fig. 1533.
Fig. 1534.
Fig. 1535.
Scale of feet
0 5 10 15 20 25 30 feet
Scale of Lbs.
2000 0 4000 8000 12000 16000 20000 Lbs
Free End
Fixed End
Span 69'0"
Rise 20'0"
1 22500 Lbs 15
20000 Lbs 10
33750
34900
30900
29750
23150
20200
13100
11000
8900
5200
2400
3600
1800
1600
1500
1200
1950
2200

Figs. 1532 to 1535.—Diagrams for Fink Truss Peculiarly Loaded.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6
7
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11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
ENLARGEMENT OF POINT OF DIAGRAM
20
17
5
11 12 27
19
18
11.12 27

Fig. 1536.

Fig. 1538.

Fig. 1537.

Figs. 1536 to 1538.—Diagram of Stresses in Large Roof Truss.

gram (Fig. 1540). The tension in the tie must then be calculated by the moments: namely, total reaction on one side, multiplied by half mean span and minus the sum of the moments of loads (that is, each load multiplied by its distance from centre line), and the remainder divided by the mean depth of truss. The result will be the tension in tie or distance 16—41. The stress diagram is shown in Fig. 1541; it should be worked to a fairly large scale, say 1 in. to each division of load. The nature of the stresses will be as shown by thick and thin lines. A stress diagram of wind pressure would be worked in the same way, but with the aid of the funicular polygon. An expansion shoe would not be necessary. The principal rafters would require calculating for the combined thrust as found by diagram, and cross strain due to purlins.

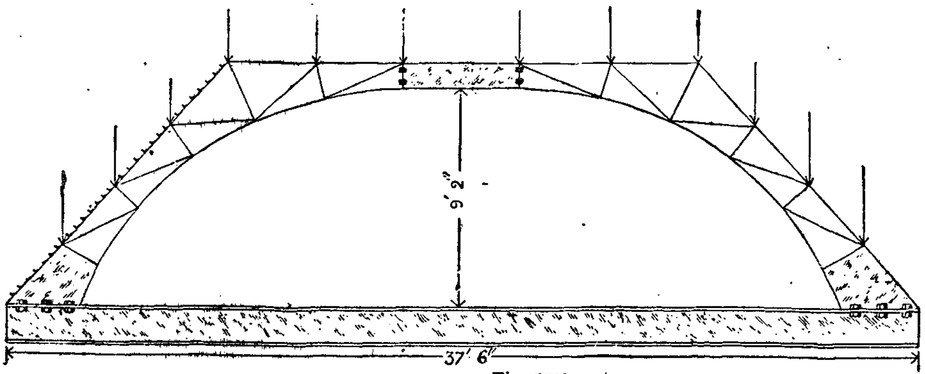


Fig. 1539

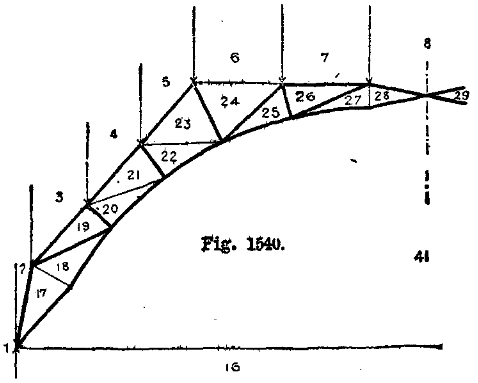


Fig. 1540.

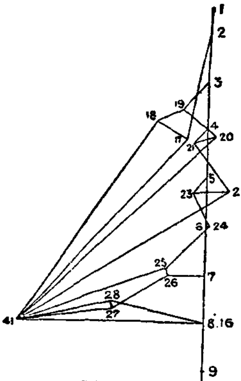


Fig. 1541.

Figs. 1539 to 1541.—Diagrams showing Stresses in Roof Truss with Plate Webs.

Steel Roof Truss with Curved Soffit.

Roofs of this character are not easily calculated, and the stresses at the lower ends in particular are somewhat indeterminate. Fig. 1542 shows the frame diagram of a roof truss with a span of 32 ft., the dotted straight lines being substituted for the curved lines to enable a reciprocal diagram to be constructed. Taking the effect of the wind on one side as included in the allowance of ½ cwt. per foot super., and the trusses as 10 ft. apart, the loads at each point of support will be as shown in Fig. 1542. In the reciprocal diagram Fig. 1543, it is assumed that all the load comes on bars 11—2, 11—10, and 19—7, 19—20, of Fig. 1542, so that 9—10 and 9—20 have no stress. In the reciprocal diagram Fig. 1544, the member 10—1 of Fig. 1542 is assumed to carry half the general load on that side of the truss, together with the whole of the load immediately above it. In the reciprocal diagram Fig. 1545, it is assumed that, the effect of the lower members being doubtful the equilibrating stresses for the remainder of the truss are

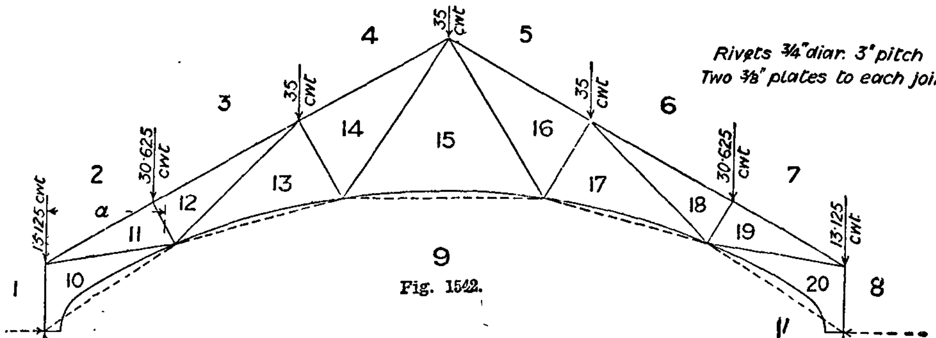

35 cwt
4
5
3
35 cwt
35 cwt
6
14
15
16
2
30·625 cwt
30·625 cwt
7
13
17
13·125 cwt
a
12
18
19
13·125 cwt
11
9
1
10
20
8
1a
96·25 cwt
96·25 cwt
8a

Fig. 1542.

Rivets ¾" diar. 3" pitch
Two ⅜" plates to each joint

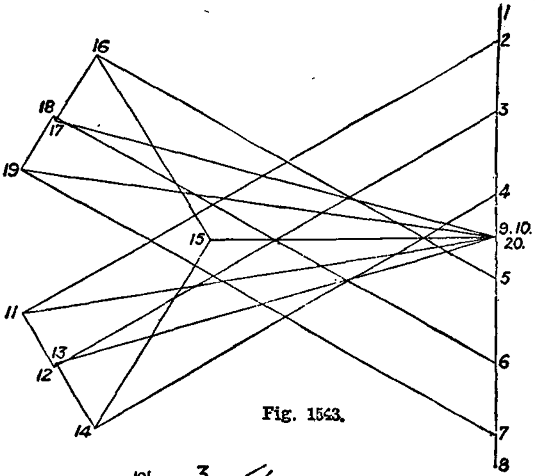

1
2
16
18
17
3
19
4
9, 10, 20.
15
5
11
13
12
6
14
7
8

Fig. 1543.

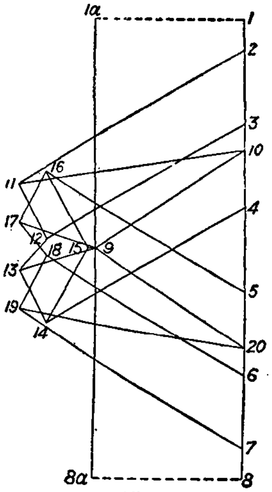

1a
1
2
3
10
11
16
4
17
12
18
15
9
13
5
19
14
20
6
7
8a
8

Fig. 1544.

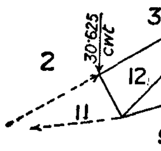

30·625 cwt
3
2
12
13
11
9

Fig. 1546.

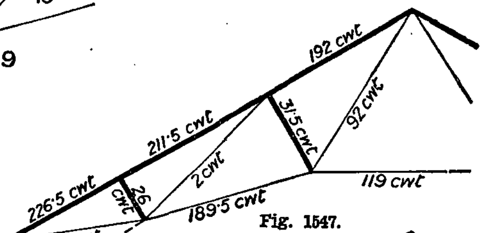

192 cwt
211·5 cwt
31·5 cwt
92 cwt
226·5 cwt
2 cwt
26 cwt
119 cwt
189·5 cwt
198 cwt
96·25 cwt

Fig. 1547.

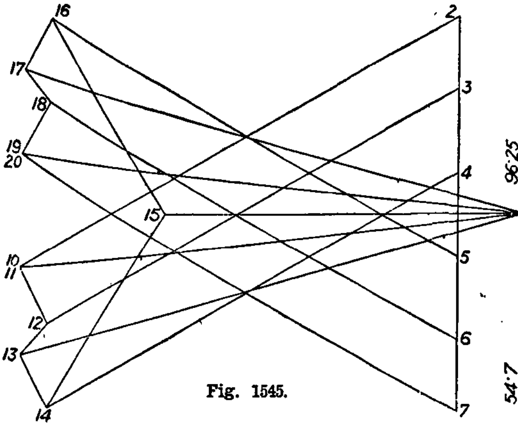

16
2
17
3
18
19
20
4
15
9
10
11
5
12
6
13
14
7

Fig. 1545.

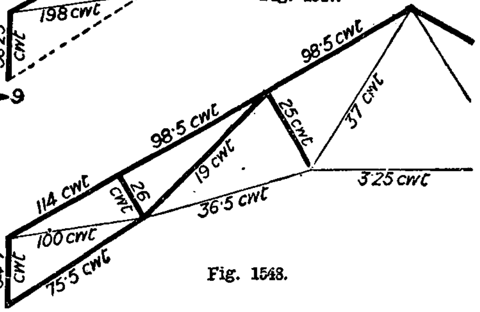

98·5 cwt
98·5 cwt
25 cwt
37 cwt
19 cwt
114 cwt
26 cwt
36·5 cwt
3·25 cwt
100 cwt
54·7 cwt
75·5 cwt

Fig. 1548.

Figs. 1542 to 1548.—Stress Diagrams for Steel Truss with Curved Soffit.

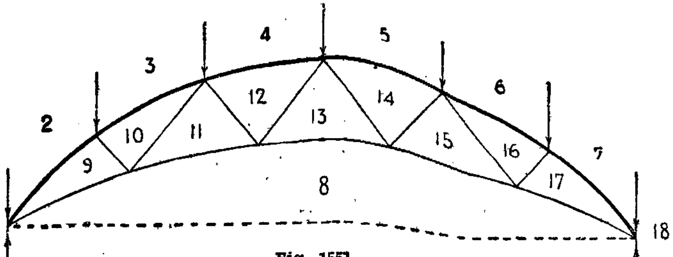

2
3
4
5
6
7
9
10
11
12
13
14
15
16
17
8
18

Fig. 1551.

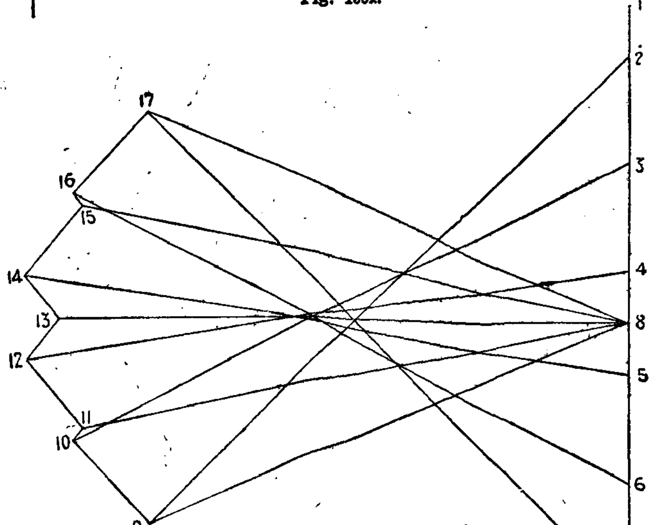

1
2
3
4
8
5
6
17
16
15
14
13
12
11
10
9

Fig. 1552.

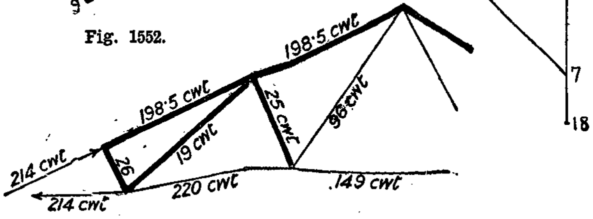

7
18
198·5 cwt
198·5 cwt
25 cwt
96 cwt
19 cwt
26
214 cwt
214 cwt
220 cwt
·149 cwt

Fig. 1549.

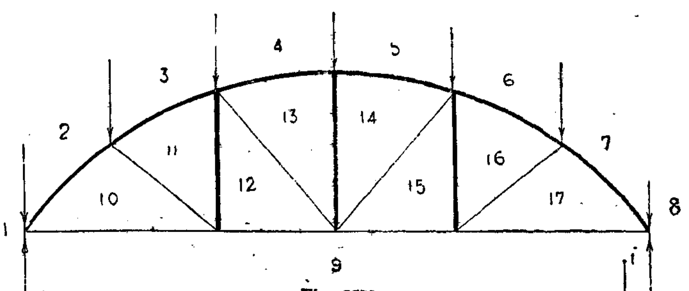

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17

Fig. 1553.

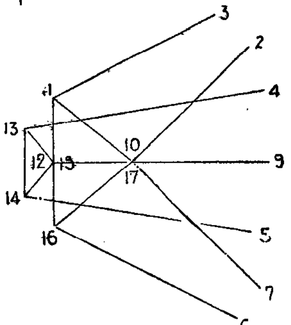

3
2
4
1
13
12
13
10
17
9
14
16
5
7
6

Fig. 1555.

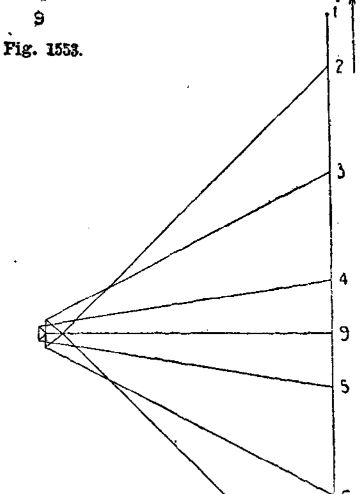


Fig. 1554.

IF CURVE BE PARABOLIC
BRACING HAS NO STRESS

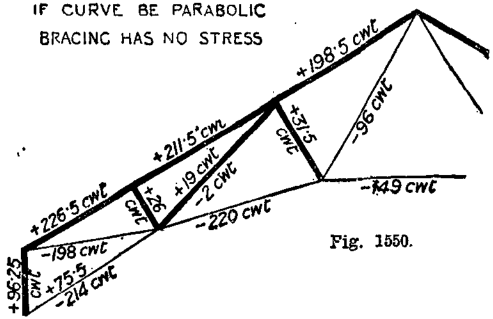

7
+198·5 cwt
+211·5 cwt
+226·5 cwt
+31·5 cwt
-96 cwt
+19 cwt
-2 cwt
+26 cwt
-198 cwt
+75·5
-214 cwt
+96·25 cwt
-220 cwt
-149 cwt

Fig. 1550.

Figs. 1549 and 1550.—Frame Diagrams of Steel Truss with Curved Soffit.

Figs 1551 to 1555.—Diagrams showing Stresses on Braced Truss with Curved Rafter.

taken as in Fig. 1546. Now, scaling the stresses from these three diagrams, we have the values shown on the corresponding Figs. 1547, 1548, and 1549. Then, collating these maxima, we have the values shown in Fig. 1550. Designing the roof from these figures, there may possibly be a little excess of strength given; but as the course of the stress is indefinite, and partly due to workmanship, it will be on the safe side. This truss, with the complete scantlings and details of joints, is described on p. 322.

Stresses on Braced Truss with Curved Rafter.

In the first case (Fig. 1551) all the members except the principal rafter are in tension, as shown by stress diagram Fig. 1552. In the second case (Fig. 1553) the principal rafter and the vertical bars will be in compression, the remainder in tension. Fig. 1555 is an enlargement to show the junctions of Fig. 1554 more clearly. If the load lines be drawn to scale, the stresses under any given loads may be measured off. If allowance is to be made for wind blowing sideways, the funicular polygon must be used.

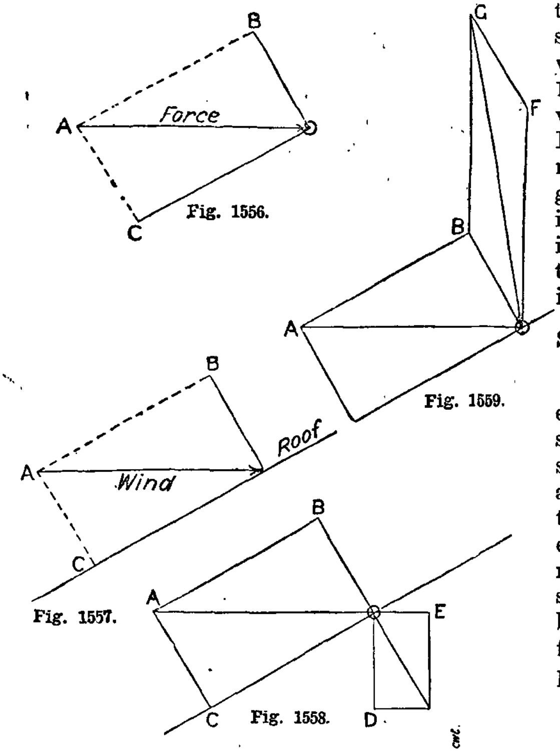

B
A
Force
C
G
F
B
A
B
Roof
A
Wind
C
B
A
E
C
D

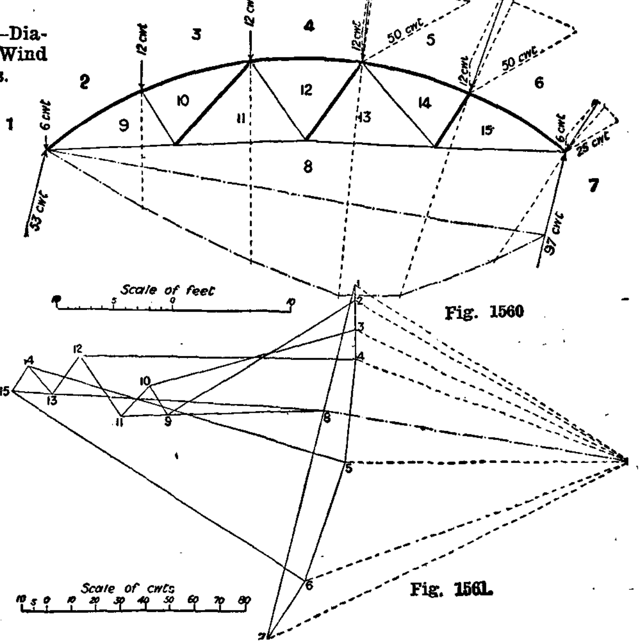
Fig. 1556.

Fig. 1557.

Fig. 1558.

Fig. 1559.

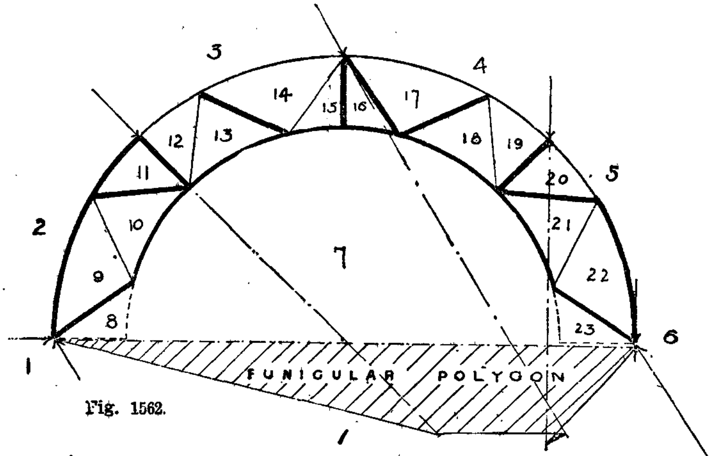
Figs. 1556 to 1559.—Diagrams showing Wind Pressure on Roofs.


12 cwt
3
12 cwt
4
12 cwt
50 cwt
5
50 cwt
12 cwt
6
2
1
6 cwt
10
9
11
12
13
14
15
6 cwt
25 cwt
8
7
33 cwt
97 cwt
Scale of feet
Fig. 1560
1
2
3
12
4
14
15
13
10
11
9
8
5
Scale of cwts
10 5 0 10 20 30 40 50 60 70 80
6
7
Fig. 1561.

Figs. 1560 and 1561.—Stresses produced by Wind Pressure on Curved Roof supported on Columns.

Pressure of Wind on Roofs.

When the wind strikes a roof at an angle, the impact is not the same as that of a force striking a body in the manner shown in the text-books of mechanics, where the force A (Fig. 1556) can be resolved into two forces B and C, and there end the matter. In the case of the wind meeting a sloping roof, the first effect is that force C slides away and force B acts with full effect at right angles to the rafters (see Fig. 1557). Then the effect of B may be resolved into two directions D and E (shown inside the roof in order to save overlapping of diagrams), as shown in Fig. 1558. The force D produces vertical load, and force E tends to push the roof off its bearings; but when the effect is required for calculating the scantlings of the truss, force B is taken and combined in a parallelogram of



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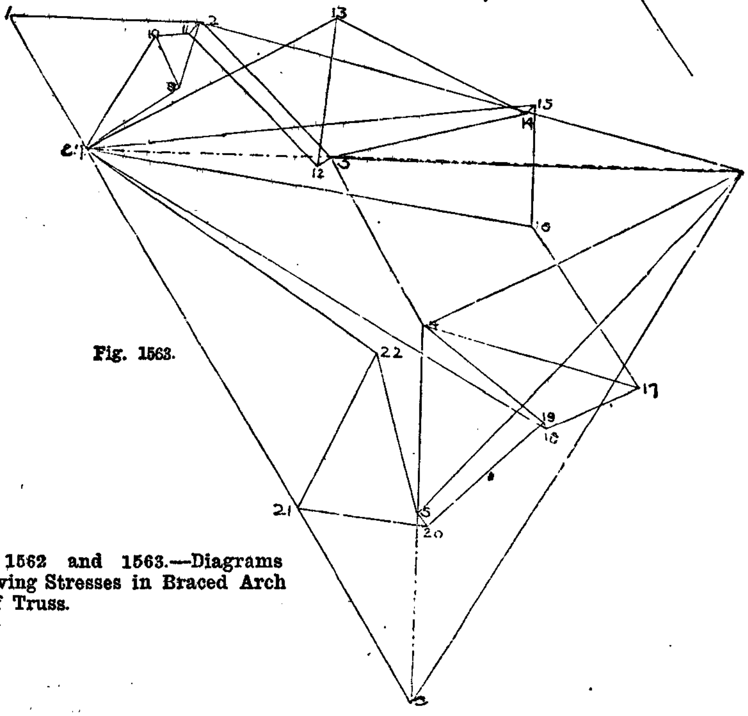
22
23
FUNICULAR POLYGON

Fig. 1562.



1
2
9
10
11
13
15
14
E
12
3
6
4
22
17
19
18
21
5
20

Fig. 1563.

Figs. 1562 and 1563.—Diagrams showing Stresses in Braced Arch Roof Truss.

forces with the weight of the roof-covering F acting at the same point, giving the final resultant G as shown in Fig. 1559.

Curved Roof supported on Columns.

For a roof as indicated in the accompanying sketch, the wind pressure acts from one side. The wind has been taken at 30° from the horizontal on one side of the roof, and then compounded with the direct load. This, it will be seen by Fig. 1560, causes all the bars inclined from the wind to be in compression and the others in tension ; but if the wind were blowing from the opposite side, these stresses would be reversed. As compression is a more trying stress, it will therefore be necessary to see that the bars are all strong enough to resist it. The columns supporting the roof cannot be braced

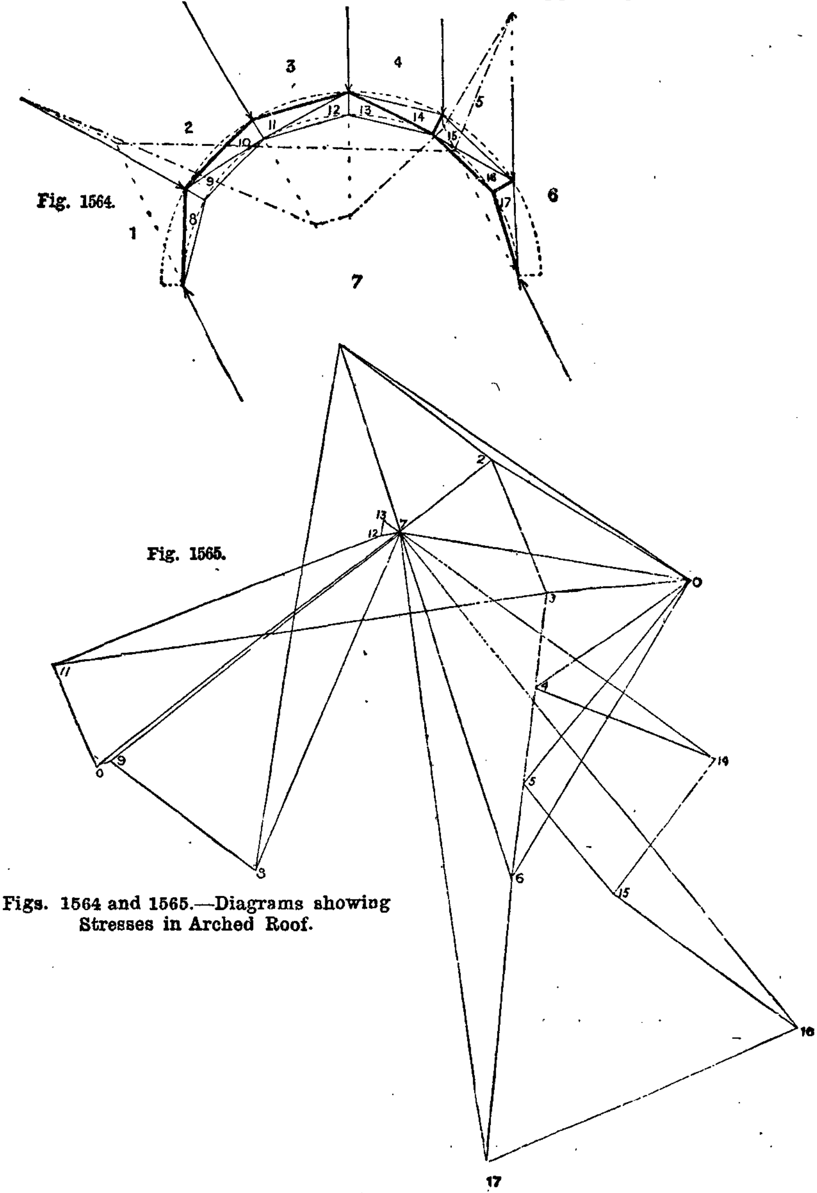


Fig. 1564.

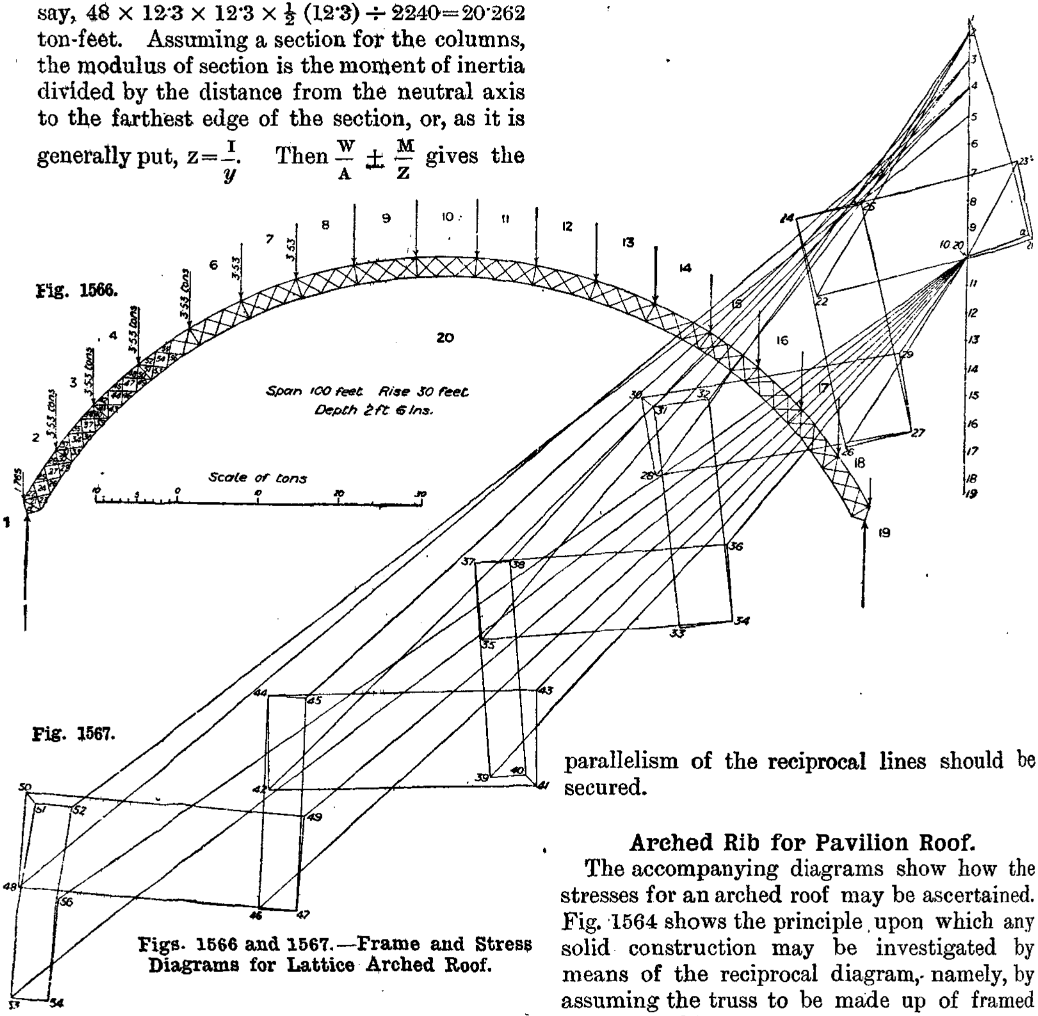
Fig. 1565.

Figs. 1564 and 1565.—Diagrams showing Stresses in Arched Roof.

to it; they must therefore be sunk in the ground, or bolted to a sufficiently massive foundation to prevent overturning. Fig. 1561 shows the stress diagram. For the bending moment on the side columns, the roof truss is of such a character that the whole force of wind must be taken by the column on the side next the wind. The bending moment will then be, say, $48 \times 12{\cdot}3 \times 12{\cdot}3 \times \frac{1}{2}(12{\cdot}3) \div 2240 = 20{\cdot}262$ ton-feet. Assuming a section for the columns, the modulus of section is the moment of inertia divided by the distance from the neutral axis to the farthest edge of the section, or, as it is generally put, $z = \frac{I}{y}$. Then $\frac{W}{A} \pm \frac{M}{Z}$ gives the maximum stress, W being the load, A the area of section, M the bending moment, and Z the modulus of section.

Fig. 1566.

Fig. 1567.



Figs. 1566 and 1567.—Frame and Stress Diagrams for Lattice Arched Roof.

Braced Arch Roof Truss.

In investigating a complex case, a simplified truss of the same type—say, for instance, the truss shown in Fig. 1562—may sometimes be substituted. Then draw the load line, and by means of a funicular polygon find the reactions 1—6, 6—5. Then the stress diagram may be completed as shown in Fig. 1563. In the actual roof the stress diagram should be worked out very carefully to a large scale on at least a full-size imperial sheet, and accurate parallelism of the reciprocal lines should be secured.

Arched Rib for Pavilion Roof.

The accompanying diagrams show how the stresses for an arched roof may be ascertained. Fig. 1564 shows the principle upon which any solid construction may be investigated by means of the reciprocal diagram, namely, by assuming the truss to be made up of framed bars. In this diagram a smaller and deeper rib is taken, to show more clearly the mode of working; in the actual case, a sheet of double elephant paper, and very neat work, will be required to get a true result. First draw the outline, as shown in Fig. 1564; dividing the rib up into triangles, show the external force

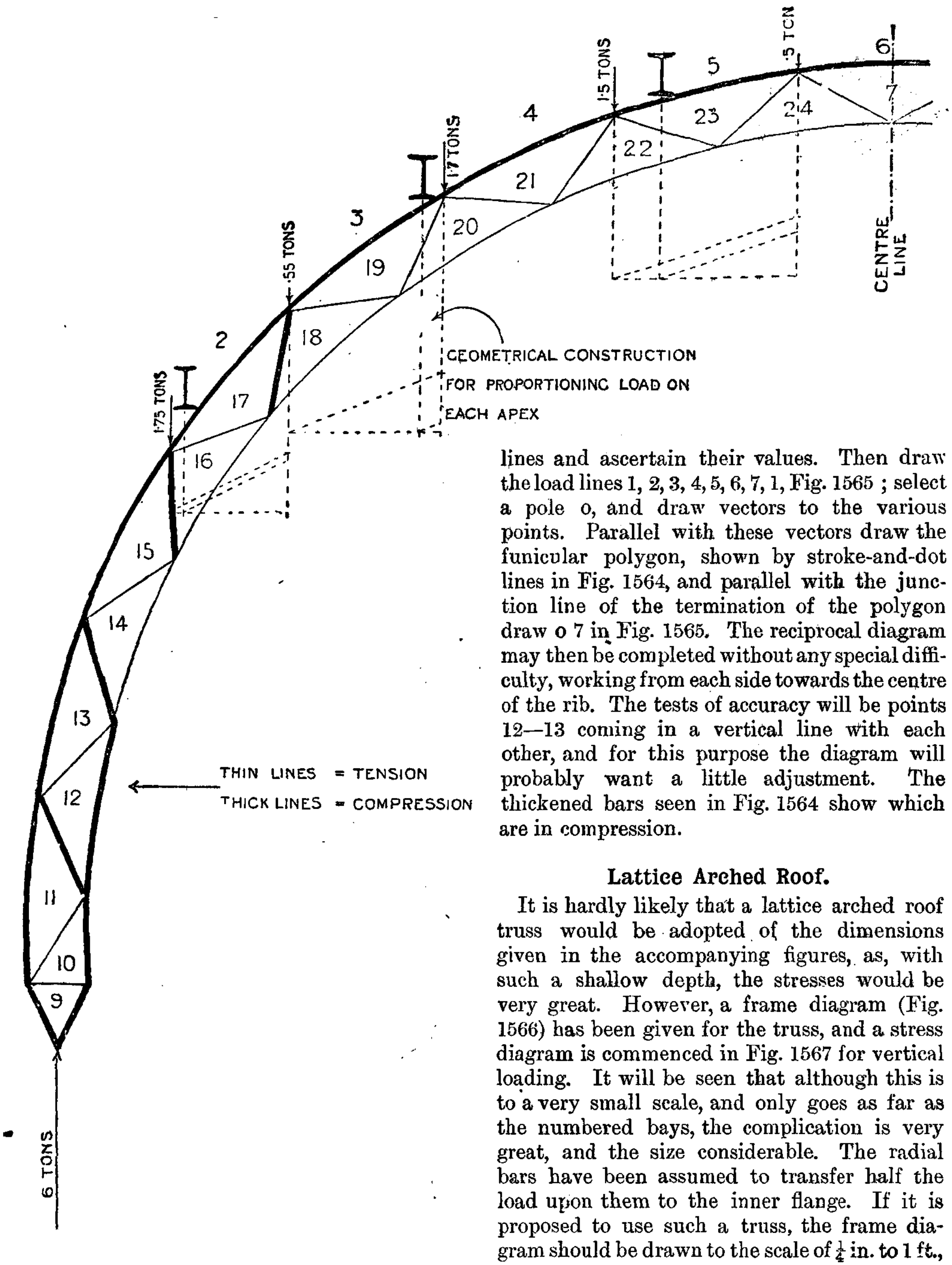

6
7
5
·5 TONS
4
1·5 TONS
24
23
22
21
1·7 TONS
20
3
·55 TONS
19
18
2
1·75 TONS
17
16
15
14
13
12
11
10
9
6 TONS
CENTRE LINE
GEOMETRICAL CONSTRUCTION FOR PROPORTIONING LOAD ON EACH APEX
THIN LINES = TENSION
THICK LINES = COMPRESSION

Fig. 1568.--Frame Diagram of Arched Roof Truss.

lines and ascertain their values. Then draw the load lines 1, 2, 3, 4, 5, 6, 7, 1, Fig. 1565 ; select a pole o, and draw vectors to the various points. Parallel with these vectors draw the funicular polygon, shown by stroke-and-dot lines in Fig. 1564, and parallel with the junction line of the termination of the polygon draw o 7 in Fig. 1565. The reciprocal diagram may then be completed without any special difficulty, working from each side towards the centre of the rib. The tests of accuracy will be points 12—13 coming in a vertical line with each other, and for this purpose the diagram will probably want a little adjustment. The thickened bars seen in Fig. 1564 show which are in compression.

Lattice Arched Roof.

It is hardly likely that a lattice arched roof truss would be adopted of the dimensions given in the accompanying figures, as, with such a shallow depth, the stresses would be very great. However, a frame diagram (Fig. 1566) has been given for the truss, and a stress diagram is commenced in Fig. 1567 for vertical loading. It will be seen that although this is to a very small scale, and only goes as far as the numbered bays, the complication is very great, and the size considerable. The radial bars have been assumed to transfer half the load upon them to the inner flange. If it is proposed to use such a truss, the frame diagram should be drawn to the scale of $\frac{1}{4}$ in. to 1 ft., and the stress diagram to a scale of at least 10 tons to 1 in., with the wind on one side. A

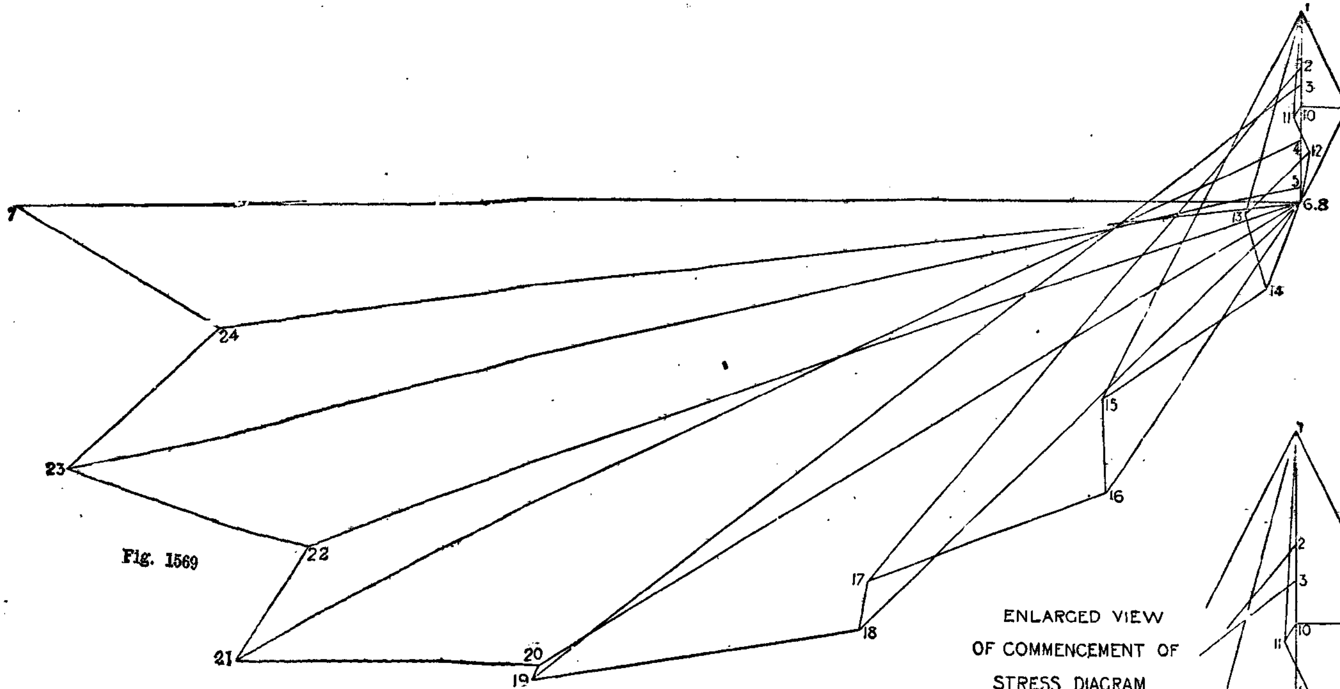

1
2
3
9
10
11
4
12
5
6.8
13
14
15
16
17
18
20
19
21
22
23
24
7

Fig. 1569

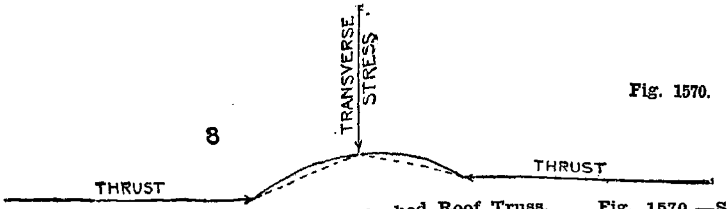
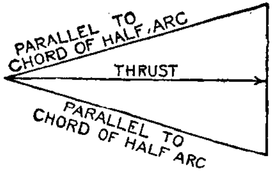

8
TRANSVERSE STRESS
THRUST
THRUST

Fig. 1570.


PARALLEL TO CHORD OF HALF ARC
THRUST
PARALLEL TO CHORD OF HALF ARC

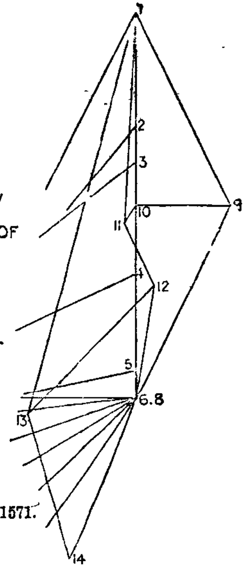

ENLARGED VIEW OF COMMENCEMENT OF STRESS DIAGRAM
1
2
3
10
11
9
4
12
5
6.8
13
14

Fig 1571.

Fig. 1569.—Stress Diagram of Arched Roof Truss.

Fig. 1570.—Stress in a Curved Member.

Fig. 1571.—Enlarged Drawing. of First Part of Stress Diagram.

sheet of double elephant paper, and very careful working, will be required.

Stresses in Arched Roof.

Figs. 1568 and 1569 show the half frame and stress diagrams of a stilted semicircular arched truss. It is not an economical form of roof for a small span, but the same method of working may be applied to a truss for any span. It should be borne in mind that the stress in a curved member tends to straighten it if in tension, and bend it further if in compression, producing thereby a transverse stress on the latter, as shown by Fig. 1570, where the thrust is put down to scale, and lines are drawn from one end parallel with the chords of the half arcs, being cut off by a vertical line through the other end. The length of this vertical line will be the measure of the transverse stress. Fig. 1571 shows an enlargement of the first part of the stress diagram.

Stresses in Hammer-Beam Roof Truss.

The hammer-beam roof truss is one of those in which the stresses depend upon circumstances outside the construction of the truss itself; like the collar-beam roof, which may have the collar either in tension, compression,

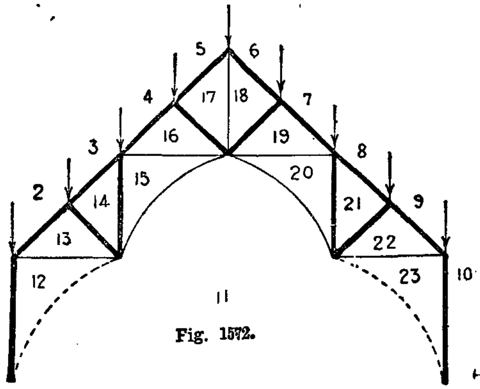

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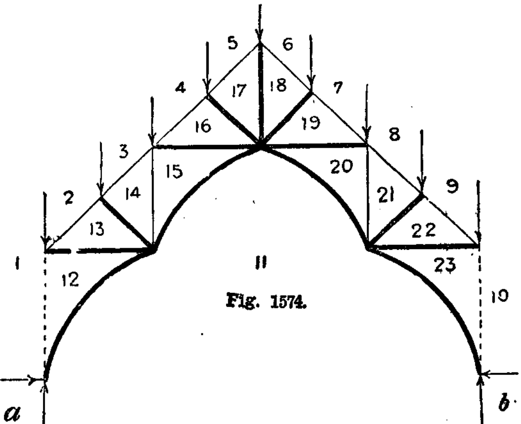
Fig. 1572.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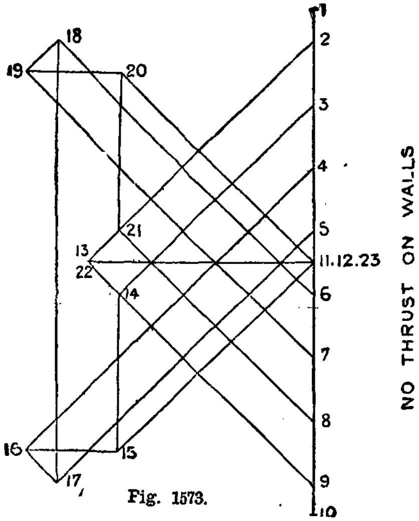
22
1
12
11
23
10
a
b

Fig. 1574.


1
18
2
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20
3
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11.12.23
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8
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15
9
17
10
NO THRUST ON WALLS

Fig. 1573.

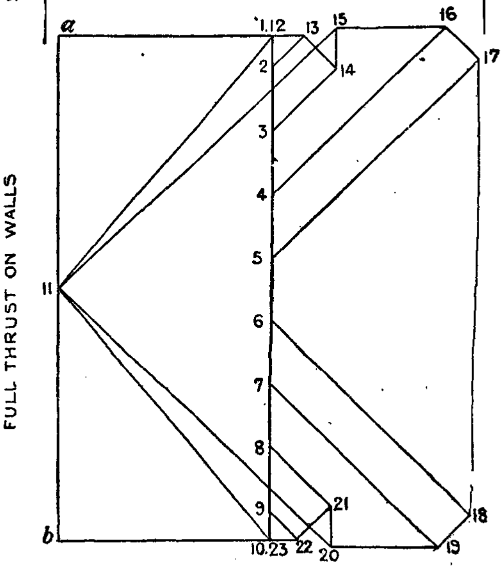
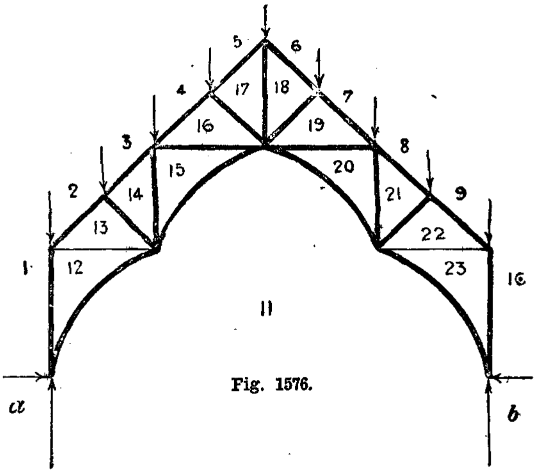

FULL THRUST ON WALLS
a
1.12
13
15
16
2
14
17
3
4
5
11
6
7
8
9
21
18
b
10.23
22
20
19

Fig. 1575.

Figs. 1572 to 1575.—Diagrams showing Stresses in Hammer-Beam Trusses.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21
22
23
a
b

Fig. 1576.

a
b
1
2
3
12
4
5
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23
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13
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15
16
17
11
20
19
18
21
22
MEDIUM THRUST ON WALLS

Fig. 1577.

Figs. 1576 and 1577.—Frame and Stress Diagrams of Hammer-Beam Truss.

or neutral, according to whether the abutments are yielding, unyielding, or yielding just sufficient to neutralise the stresses. If the walls are assumed to be capable of resisting the full thrust, as such walls certainly ought to be, the nature of the stresses will be reversed in many of the members. Figs. 1572 and 1573 show frame and stress diagrams, assuming that the walls are incapable of doing more than carry the direct weight. Figs. 1574 and 1575 the same for walls firmly buttressed and unyielding; Figs. 1576 and 1577 the same for an intermediate case, where the wall piece or the wall behind it carries one-fourth of the total load. A similar roof is shown in Fig. 1578, in which the scantlings have been determined from the stresses. Fig. 1579 shows how the purlins are fitted into the principal rafters. Fig. 1580 shows the modification of truss to enable a stress diagram to be drawn, and Fig. 1581 shows the stress diagram. The stresses will vary in nature and amount with the rigidity of the truss and the stability of the walls, and no very precise estimate can be formed, but to be on the safe side allowance should be made for partial failure of any part.

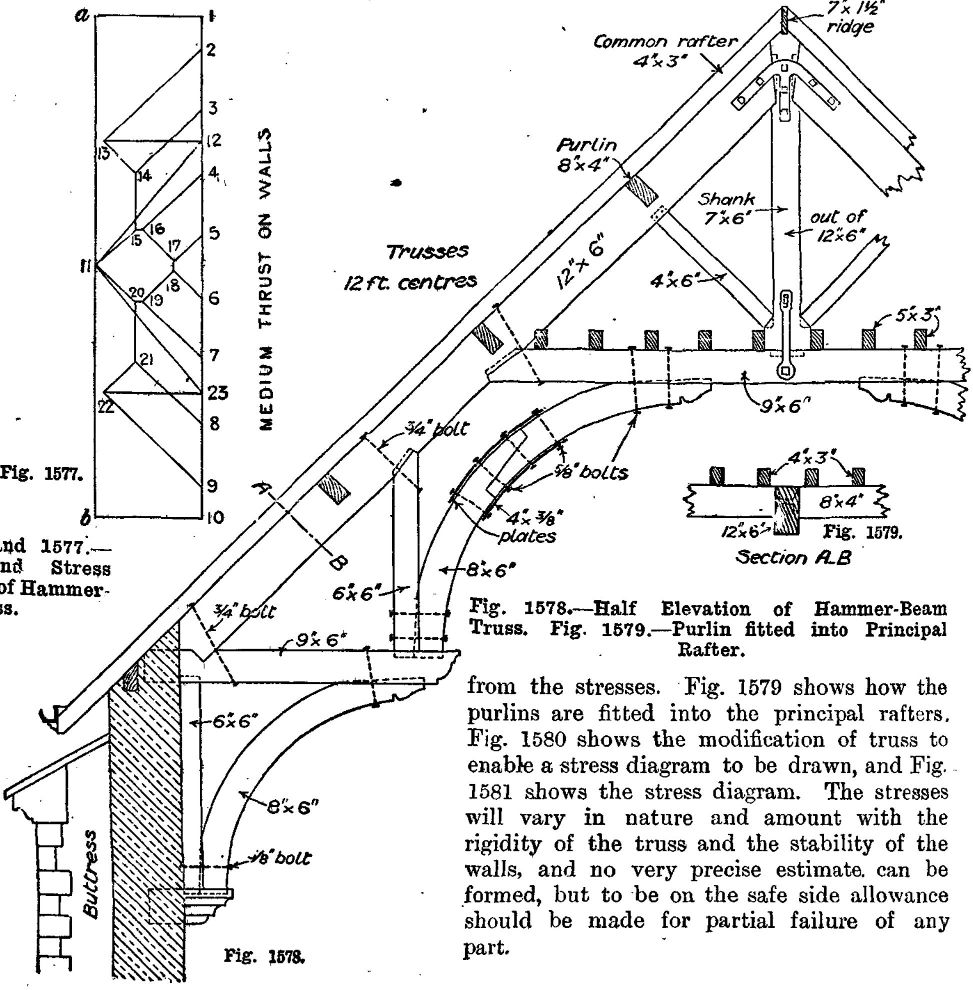

7"x 1½" ridge
Common rafter 4"x 3"
Purlin 8"x 4"
Shank 7"x 6"
out of 12"x 6"
Trusses 12 ft. centres
12"x 6"
4"x 6"
5"x 3"
9"x 6"
¾" bolt
⅝" bolts
4"x ⅜" plates
8"x 6"
6"x 6"
A
B
¾" bolt
9"x 6"
6"x 6"
8"x 6"
⅞" bolt
Buttress

Fig. 1578.

4"x 3"
8"x 4"
12"x 6"
Section A.B

Fig. 1579.

Fig. 1578.—Half Elevation of Hammer-Beam Truss. Fig. 1579.—Purlin fitted into Principal Rafter.

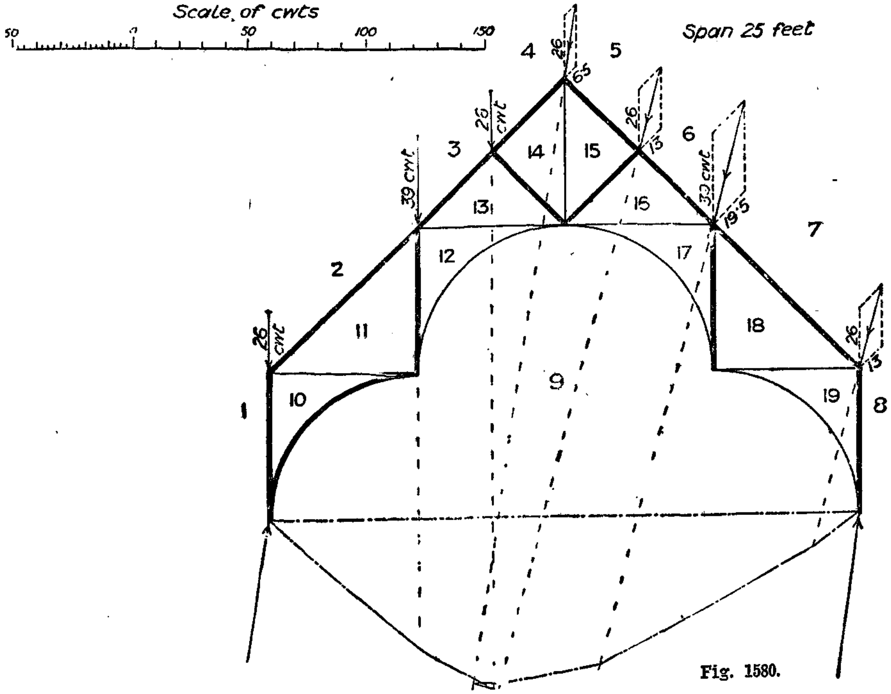
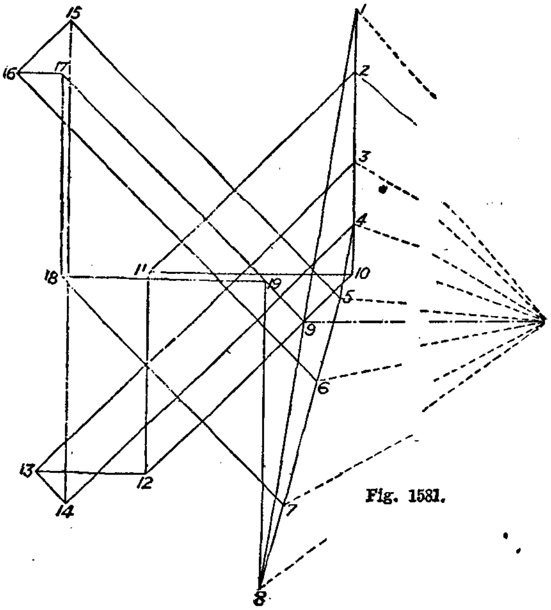

Scale of cwts
50
0
50
100
150
Span 25 feet
26 cwt
39 cwt
26 cwt
33 cwt
26
6.5
13
19.5
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19

Fig. 1580.

Figs. 1580 and 1581.—Frame and Stress Diagrams of Hammer-Beam Truss.

Directions of Reactions under a Roof Truss.

The directions of the reactions under a roof truss depend practically on several causes, such as the side on which the wind comes, the comparative stiffness of the bearing, the friction of the ends of the truss on the two walls, etc. These disturbing elements, however, cannot be determined unless special provision is made by bolting one side down and leaving the other side free or resting it on rollers. Taking a simple case as an example, let Fig. 1582 represent a roof truss under the action of the ordinary vertical load, and with the wind on one side. There are three conditions for which the stresses may be determined: namely, Fig. 1582, where both abutments are assumed to resist equally; Fig. 1583, where the truss is bolted down on the side next the wind, and is free to move on the other side; and Fig. 1584, where the truss is bolted down on the side away from the wind, being free to move on the side next the wind. Fig. 1582 is the normal case, because the wind may come from either side in practice, and both sides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19

Fig. 1581.

are alike held by friction. In Fig. 1583, as will be seen, the action of the wind causes the roof to spread, and in Fig. 1584 the roof tends to contract or double up. The inclined reactions may, by parallelogram of forces, be resolved into a vertical load and a horizontal thrust resisted by friction, or by bolting. This description will enable the student to apply the principles to any required case. As a matter of fact, the stresses in any given roof may vary between the extremes indicated by the diagrams.

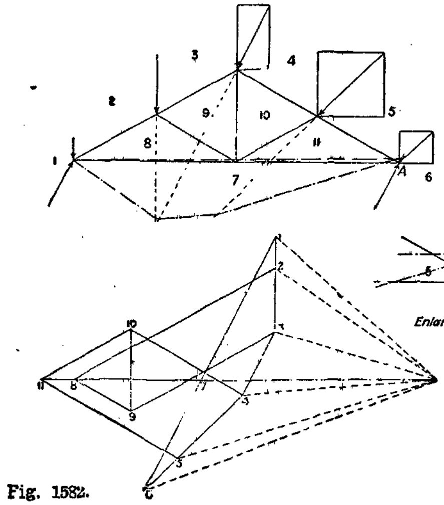
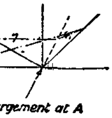


Fig. 1582.



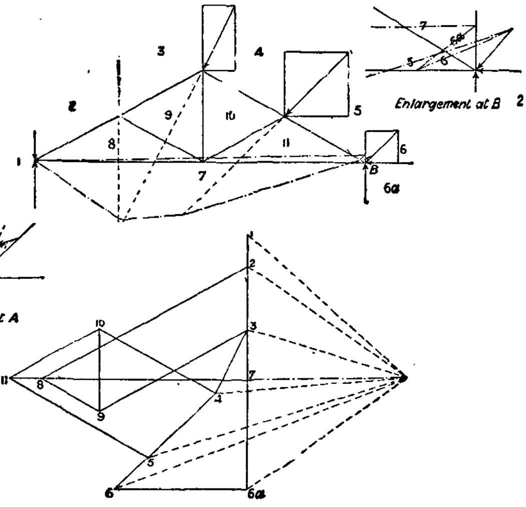


Fig. 1583.

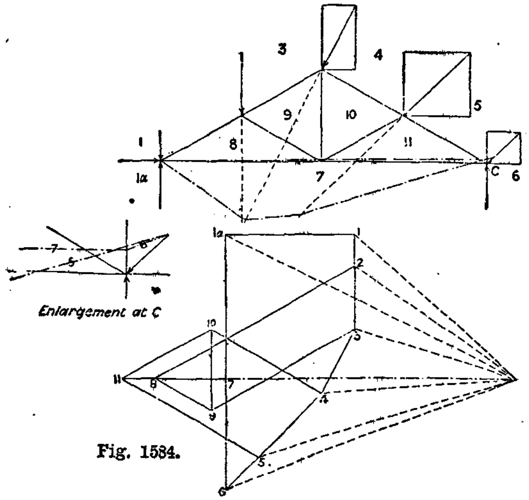


Fig. 1584.

Figs. 1582 to 1584.—Diagrams showing Direction of Reactions under Roof Truss.

Roof Truss with Wind Load and Hanging Loads.

The stresses in the roof truss (Fig. 1585) are required. This is rather an awkward diagram to work out, but there are two or three methods by which the stress diagram may be reached. Draw the frame diagram Fig. 1585, putting dotted lines 8—9 and 11—1 for the approximate reactions. Draw the load line Fig. 1586 as far as 7—8, and then a difficulty occurs, because the direction of 8—9 is unknown. The difficulty can be avoided by omitting the consideration of these two latter forces for the present. Join 8—1, Fig. 1586, for the temporary direction of 8—9 and 11—1 on the frame diagram, then construct the funicular polygon as shown, and find from it the position of point X on 8—1 in the stress diagram; call this point 10, then set 10—11 downwards and 10—9 upwards, and join 8—9 and 11—1. The diagram Fig. 1586 can then be completed, and the reactions corrected in Fig. 1585. Another method would be to find the reactions by means of a substituted frame for the truss given, the original truss then being reverted to for the stress diagram.

Queen-Post Truss with Wind on One Side.

A queen-post truss with the wind acting on one side is a deformable structure—that is, it is incomplete as a truss and depends for stability upon its stiffness. This is owing to

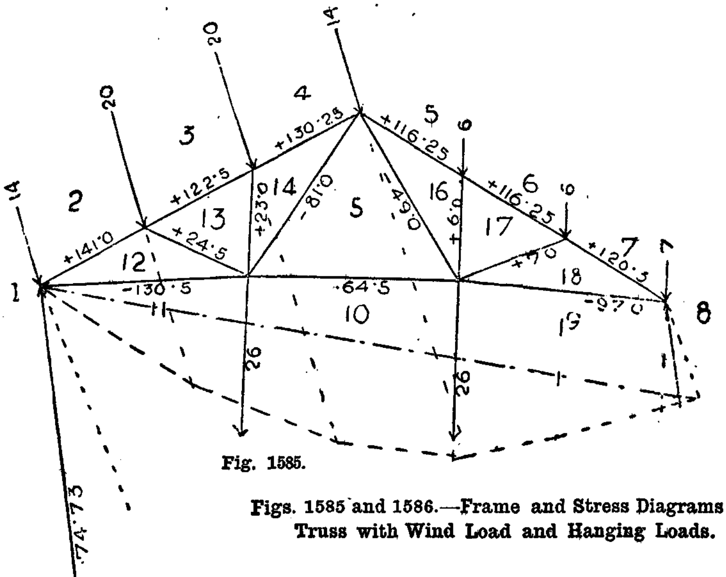
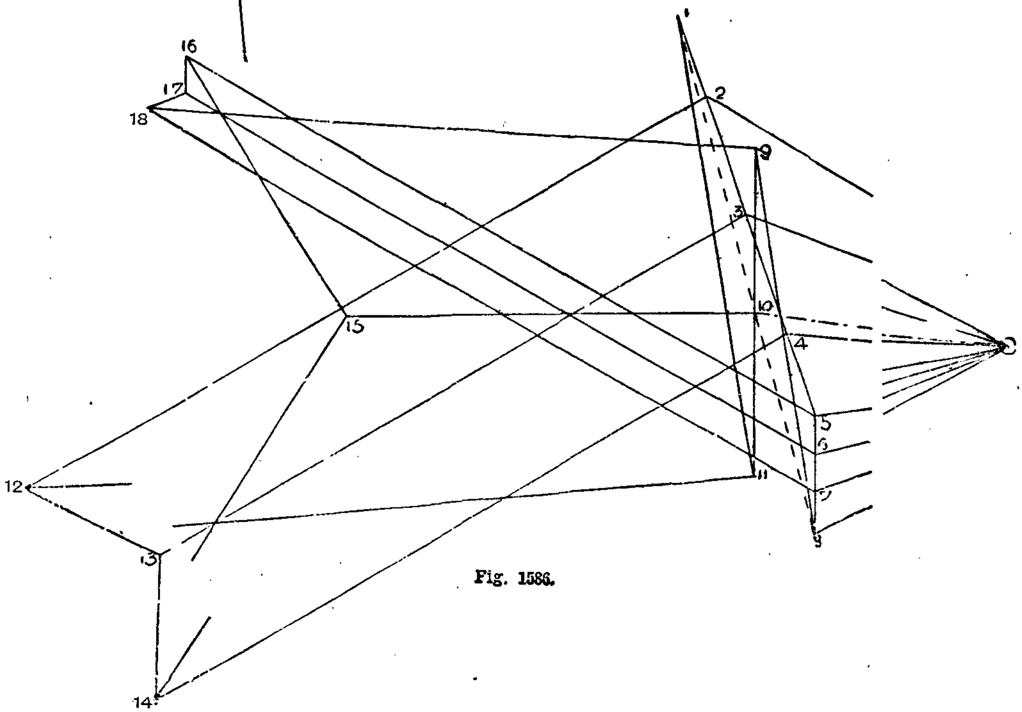

14
20
20
14
6
6
7
1
2
3
4
5
6
7
8
+141·0
+122·5
+130·25
+116·25
+116·25
+120·3
12
13
14
5
16
17
18
+24·5
+23·0
-81·0
-49·0
+6·9
+7·0
-130·5
-64·5
-97·0
11
10
19
26
26
·74·73

Fig. 1585.

Figs. 1585 and 1586.—Frame and Stress Diagrams of Roof Truss with Wind Load and Hanging Loads.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11
12
13
14
15
16
17
18
0

Fig. 1586.

the quadrangular space between the queen posts, and the effect is to cause cross strains upon the tie-beam, tending to bend it downwards at the foot of the queen post on the side the wind comes, and to bend it upwards at the foot of the queen post on the side opposite the wind. In constructing the stress diagram, these bending forces have to be equilibrated by imaginary forces shown under the truss in Fig. 1591. It is a rather difficult case, and is avoided by all, or nearly all, the text-books. By the method shown herewith the difficulties are removed, and a practically correct estimate of the stresses is obtained. Draw the frame diagram of truss as in Fig. 1587, add the vertical force lines representing the weight of truss and load upon it, then draw horizontal force lines representing the wind, and, by construction, find its effect upon the roof by drawing lines from its extremities parallel and perpendicular

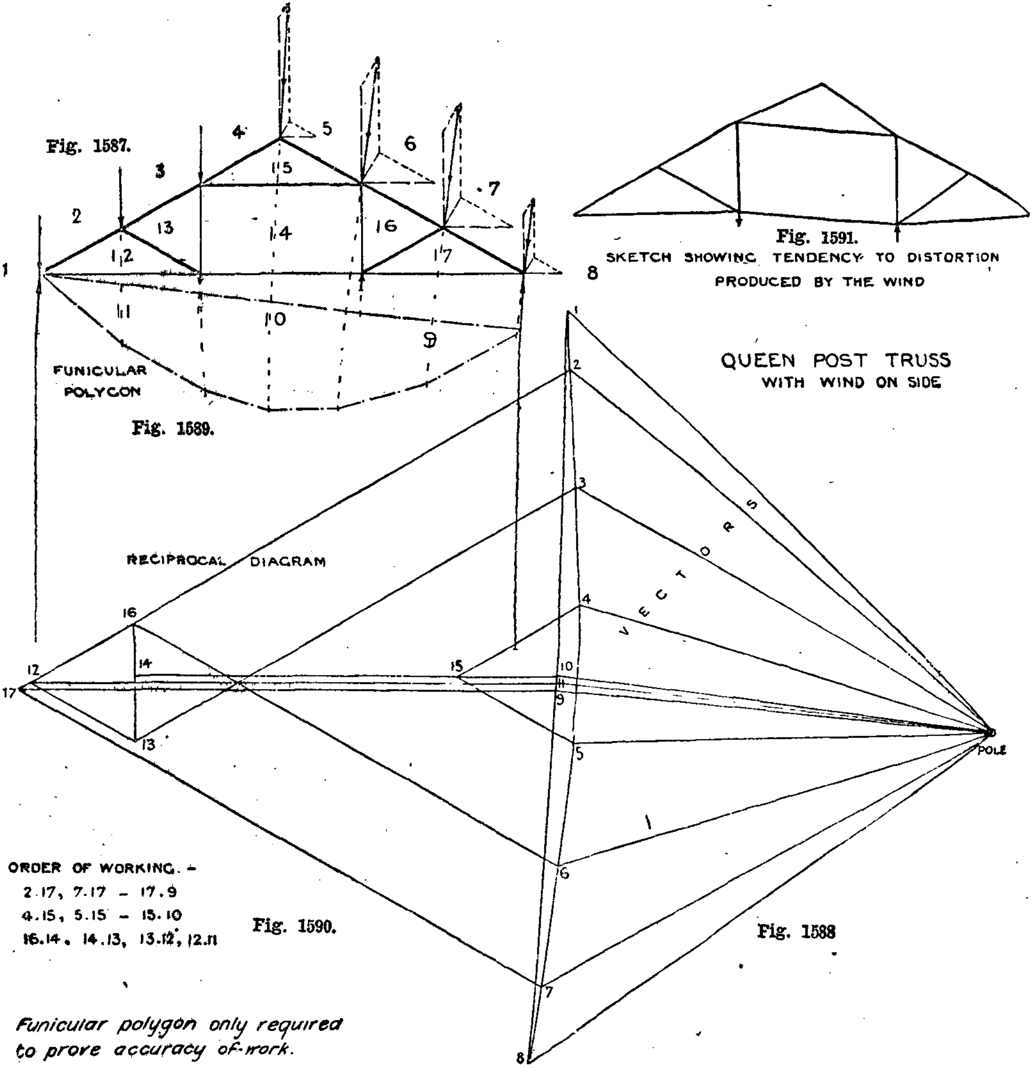
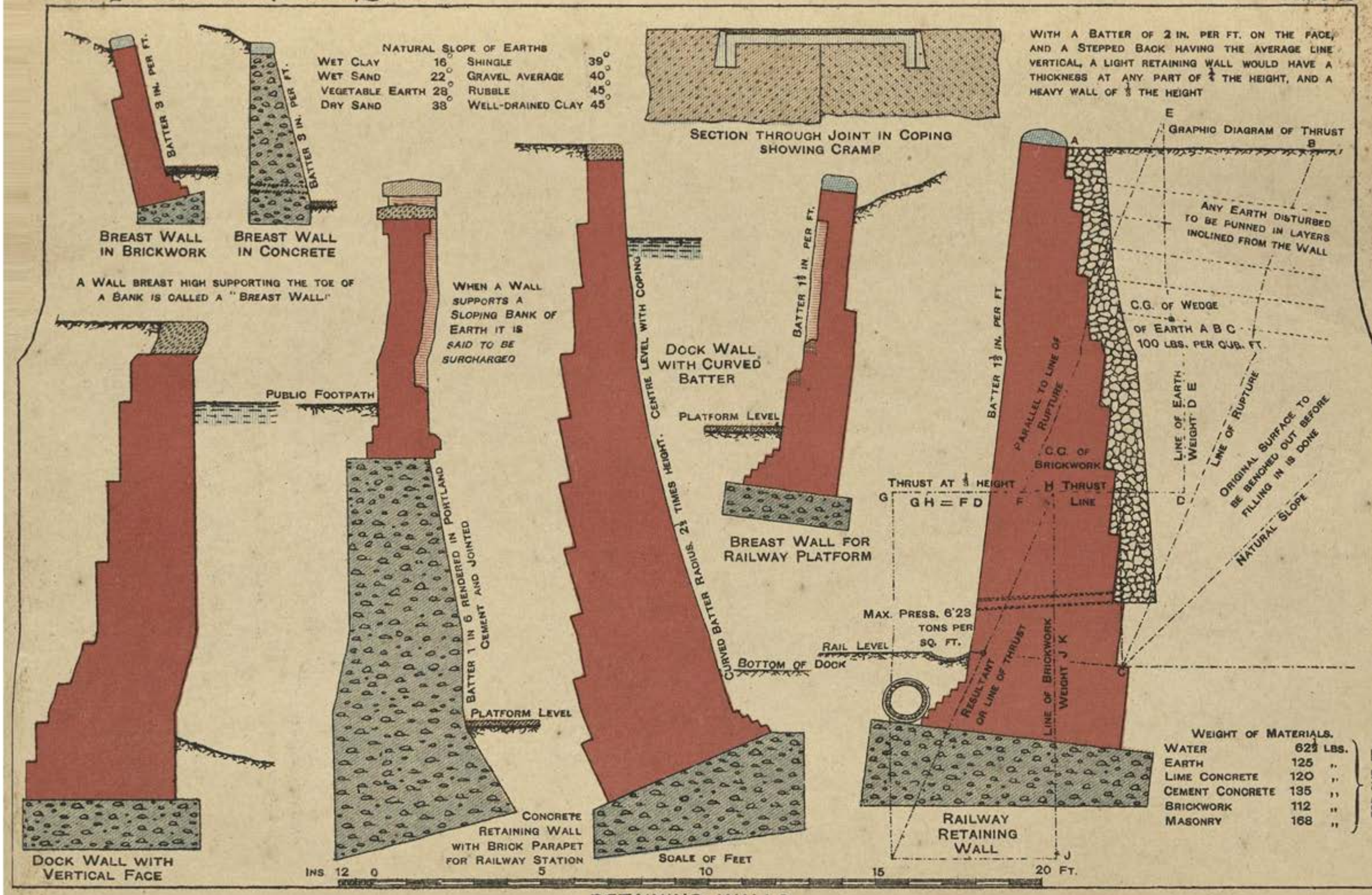


Fig. 1587.
1
2
3
4
5
6
7
8
1,2
13
1,4
1,5
16
1,7
1,1
1,0
9
FUNICULAR POLYGON
Fig. 1589.
Fig. 1591.
SKETCH SHOWING TENDENCY TO DISTORTION PRODUCED BY THE WIND
QUEEN POST TRUSS WITH WIND ON SIDE
RECIPROCAL DIAGRAM
VECTORS
POLE
12
17
16
14
13
15
10
11
9
Fig. 1588
ORDER OF WORKING. –
2.17, 7.17 – 17.9
4.15, 5.15 – 15.10
16.14, 14.13, 13.12, 12.11
Fig. 1590.
Funicular polygon only required to prove accuracy of work.

Figs. 1587 to 1591.—Diagrams showing Stresses in Queen-Post Truss.


Batter 3 in. per ft.
Breast Wall in Brickwork
Batter 3 in. per ft.
Breast Wall in Concrete
Section through Joint in Coping showing Cramp
Dock Wall with Vertical Face
Public Footpath
Batter 1 in 6 rendered in Portland Cement and jointed
Platform Level
Concrete Retaining Wall with Brick Parapet for Railway Station
Centre level with coping
Curved batter radius, 2½ times height.
Dock Wall with Curved Batter
Bottom of Dock
Batter 1½ in. per ft.
Platform Level
Breast Wall for Railway Platform
E
Graphic Diagram of Thrust
A
B
Any earth disturbed to be punned in layers inclined from the wall
C.G. of Wedge
of Earth A B C
100 lbs. per cub. ft.
Batter 1½ in. per ft
Parallel to line of rupture
C.G. of Brickwork
Line of Earth Weight D E
Line of Rupture
Original surface to be benched out before filling in is done
Natural Slope
Thrust at ⅓ height
G
GH = FD
F
H
Thrust Line
D
Max. Press. 6'23 tons per sq. ft.
Rail Level
Resultant or line of thrust
Line of Brickwork Weight J K
C
J
Railway Retaining Wall
Ins 12
0
5
10
15
20 Ft.
Scale of Feet

A wall breast high supporting the toe of a bank is called a "Breast Wall."

When a wall supports a sloping bank of earth it is said to be surcharged

With a batter of 2 in. per ft. on the face, and a stepped back having the average line vertical, a light retaining wall would have a thickness at any part of ¼ the height, and a heavy wall of ⅓ the height

Natural Slope of Earths

Wet Clay	16°	Shingle	39°
Wet Sand	22°	Gravel, average	40°
Vegetable Earth	28°	Rubble	45°
Dry Sand	38°	Well-drained Clay	45°

Weight of Materials.

Water	62½ lbs.	per cub. ft
Earth	125 ,,	
Lime Concrete	120 ,,	
Cement Concrete	135 ,,	
Brickwork	112 ,,	
Masonry	168 ,,	

RETAINING WALLS.

to the roof plane. The former line represents the amount of pressure slipping off, and the latter line the actual force on the roof. This force, normal to the roof, must now be combined with the vertical load by the parallelogram of forces, and the diagonal, shown by thick line and arrow-head in Fig. 1587, gives the resultant of the combined forces. Now draw the load line 1 to 8 in Fig. 1588, and join the extremities. The line 8—1 represents the reactions of the supports, and usually the funicular polygon, as in Fig. 1589, would be employed to find the amount of each; but there is a better method available. From point 2 in Fig. 1588 draw a ine parallel to 2—12 in Fig. 1587, and of indefinite length, and from point 7 in Fig. 1588 draw a line parallel to 7—17 in Fig. 1587. These two lines will intersect in point 17 of Fig. 1590, and a horizontal line from 17 to cut 8—1 will give point 9. In the same way from points 4 and 5 in Fig. 1588 draw lines parallel to 4—15 and 5—15 in Fig. 1587 to intersect in point 15 of Fig. 1590. A horizontal line through this point will give point 10. Then draw 16—14, 14—13, 13—12, 12—11 in Fig. 1590. It will now be seen that the value of the reactions is found, and also the value of the cross strains 9—10 and 10—11. The remainder of the reciprocal diagram can now be drawn without difficulty, and all the stresses measured off. To prove that this is a correct working, select any pole o (Fig. 1588), and draw the corresponding funicular polygon (Fig. 1589), and the fact of its accurately closing shows that all the forces have been accounted for.

Another Case.—Fig. 1592 shows the frame diagram of a queen-post truss with wind on one side. Fig. 1593 shows the stress diagram for the truss. The trusses are assumed to be 12 ft. apart, the dead load on truss 30 lb. per foot super., and the wind acting horizontally 30 lb. per foot super. The dead load and wind acting on the apex are assumed to be taken by the common rafters. Some difficulty will probably be found in drawing the stress diagram for this, unless note be taken of the fact that a queen-post truss is deformable, and that the resistance to the transverse stresses 8—9 and 9—10 on the tie-beam keeps it in shape. These stresses are of unknown amount until the stress diagram is set out, which should be done as follows: Draw the load 1 to 7, and join 7—1, which will give the direction of reactions and virtual transverse loads required to put the truss in equilibrium. From 2 draw 2—11, and produce it to meet 6—15; point 11 will be somewhere in the line, but cannot yet be determined. Draw 15—8 to cut line 7—1 in point 8. Then draw horizontal line from 4 to pass through 13 and 9, point 9 being on line 7—1. Note that point 14 must be on 15—2 to be parallel with 15—14 of frame diagram, also that point 13 must be in the same horizontal line as 4 and 9 and in the same vertical line as 14; this can only happen when 13 and 14 are at the same point. Next draw 13—12, 3—12, then 12—11, 2—11, and finally 11—10 to cut 7—1 in 10. A funicular polygon may now be drawn to test the work.

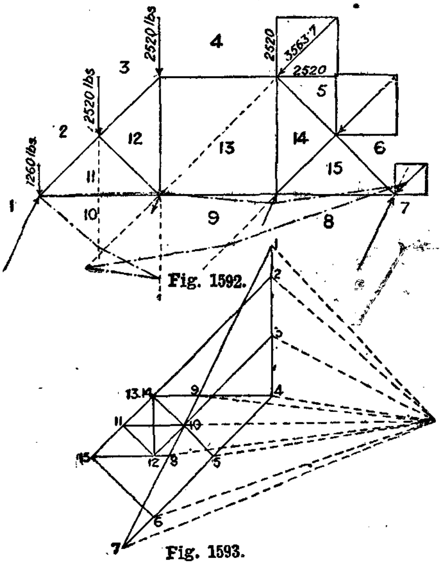


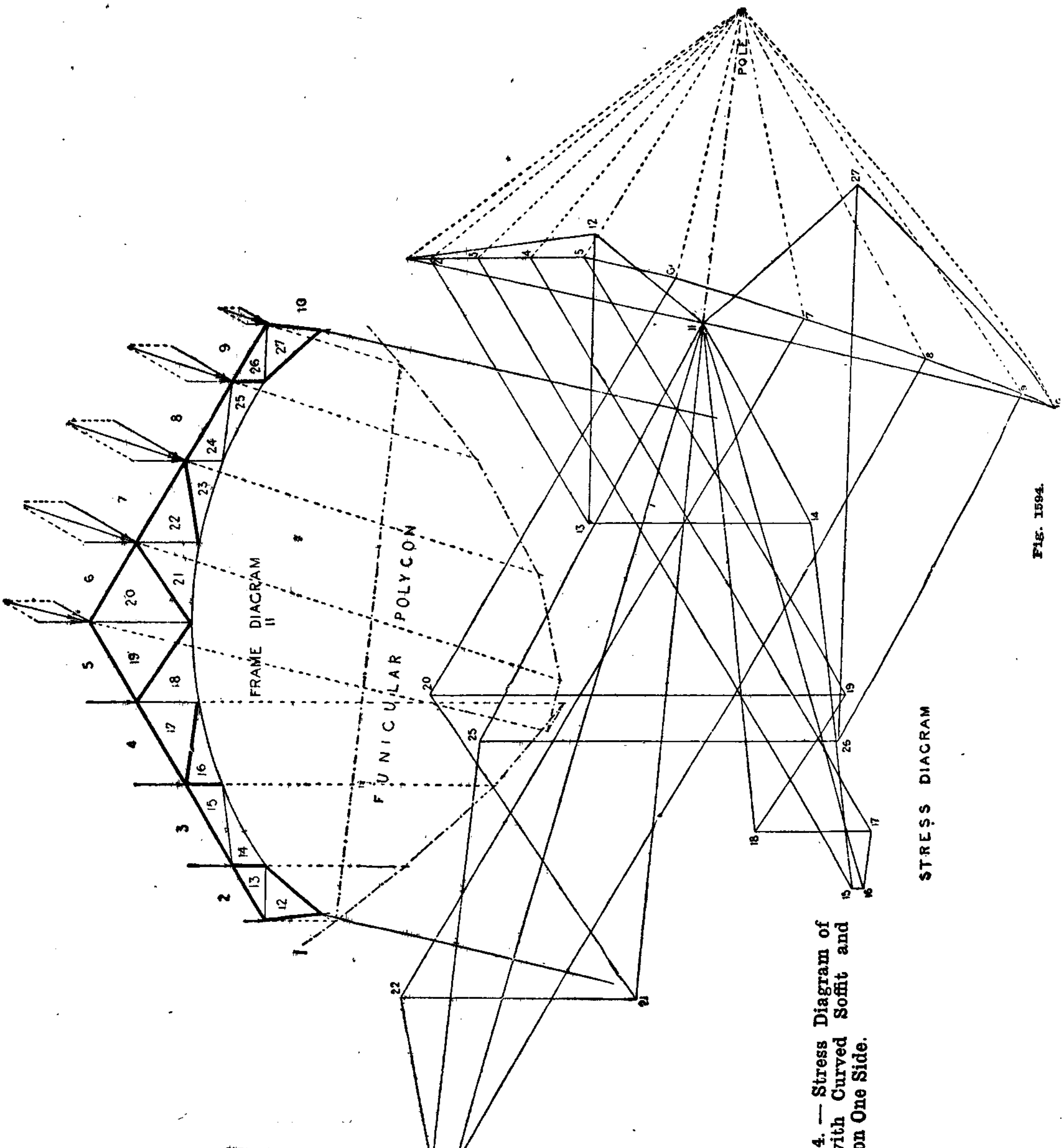
Fig. 1592.

Fig. 1593.

Figs. 1592 and 1593.—Frame and Stress Diagrams for Queen-Post Truss.

Roof with Curved Soffit and Wind on One Side.

Fig. 1594 shows the frame diagram, and Fig. 1595 the actual end of the truss. In the accompanying stress diagram Fig. 1594, the vertical load on the truss has been assumed at 16,890 lb., including the weight of the truss, and the wind as producing an additional force of 36 lb. per foot superficial on one side normal to the roof plane. There is a slight error in the closing


POLE
FRAME DIAGRAM
FUNICULAR POLYGON
STRESS DIAGRAM
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
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25
26
27

Fig. 1594.

Fig. 1594. — Stress Diagram of Roof with Curved Soffit and Wind on One Side.

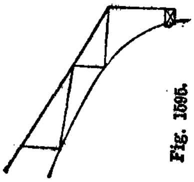

Fig. 1595.

Fig. 1595.—Actual End of Roof Truss.

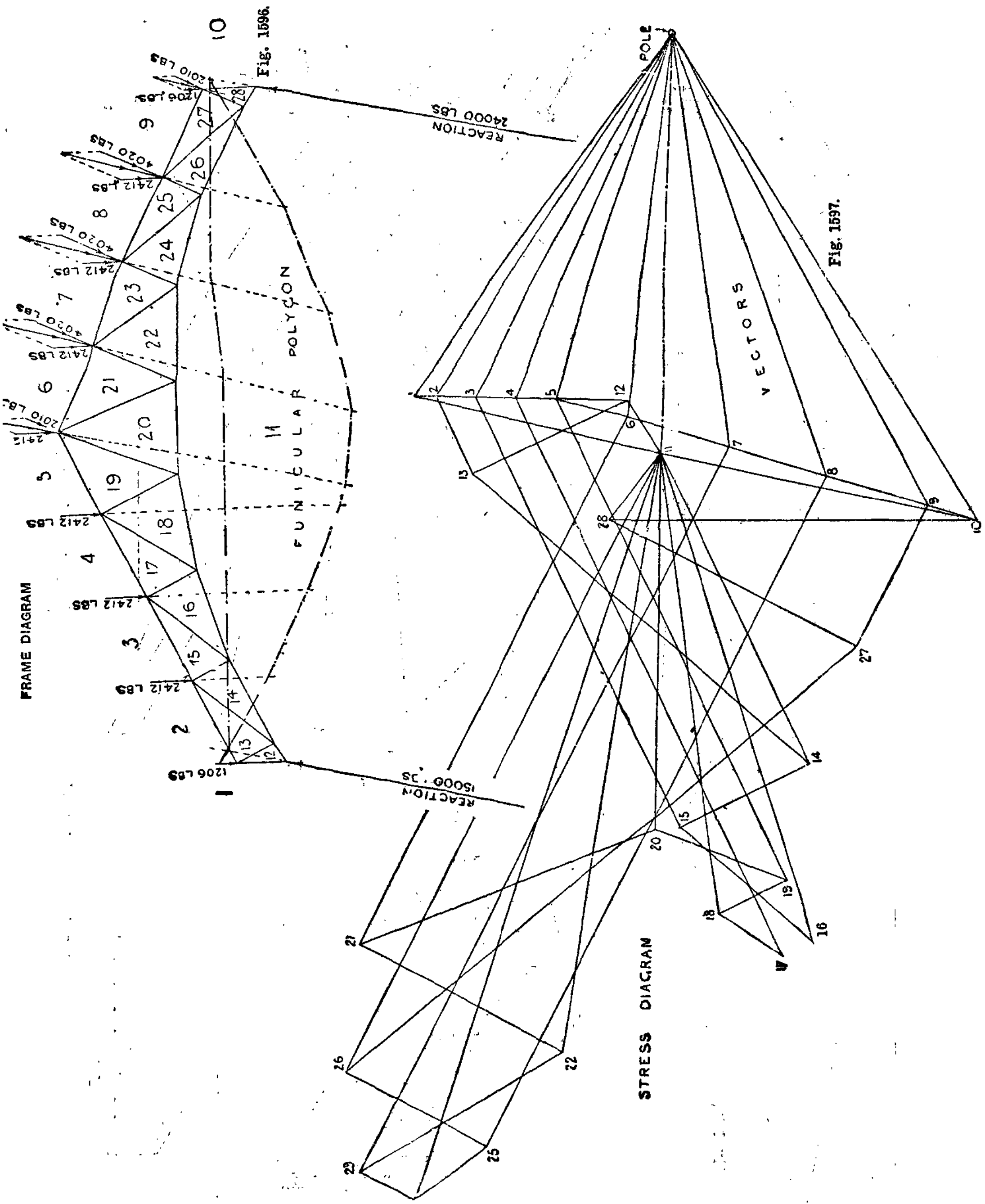
Fig. 1596.
Fig. 1597.
FRAME DIAGRAM
FUNICULAR POLYGON
STRESS DIAGRAM
VECTORS
POLE
REACTION 24000 LBS.
REACTION 15000 LBS.
1206 LBS
2010 LBS
2412 LBS
4020 LBS
2412 LBS
4020 LBS
2412 LBS
4020 LBS
2010 LBS
2412
2412 LBS
2412 LBS
2412 LBS
1206 LBS
1
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3
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Figs. 1596 and 1597.—Frame and Stress Diagrams of Steel Truss, 80-ft. Span.

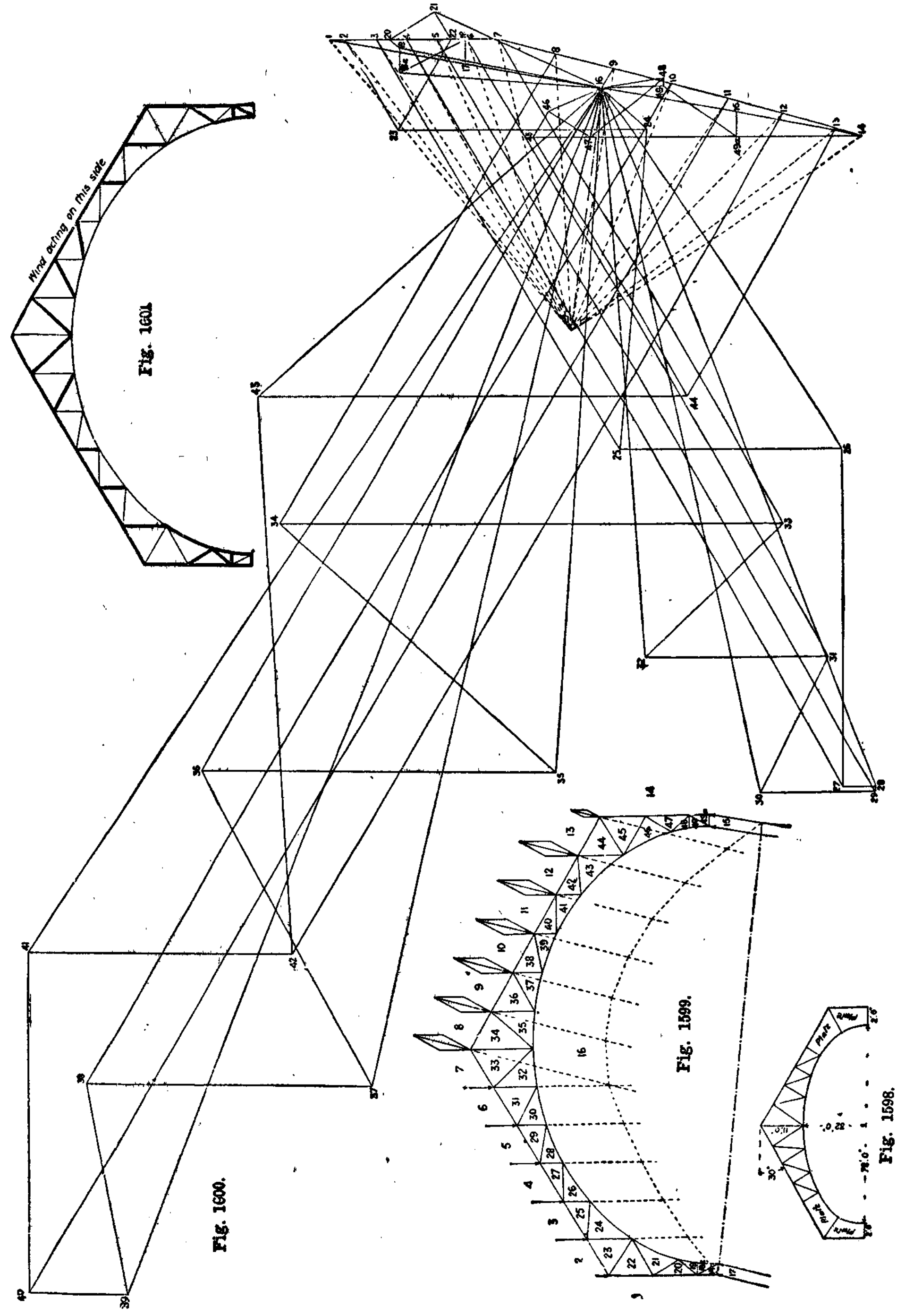
Fig. 1601.
Wind acting on this side
Fig. 1600.
Fig. 1599.
Fig. 1598.
Plate
Plate
30°
16
14
41
42
43
44
34
33
35
36
37
38
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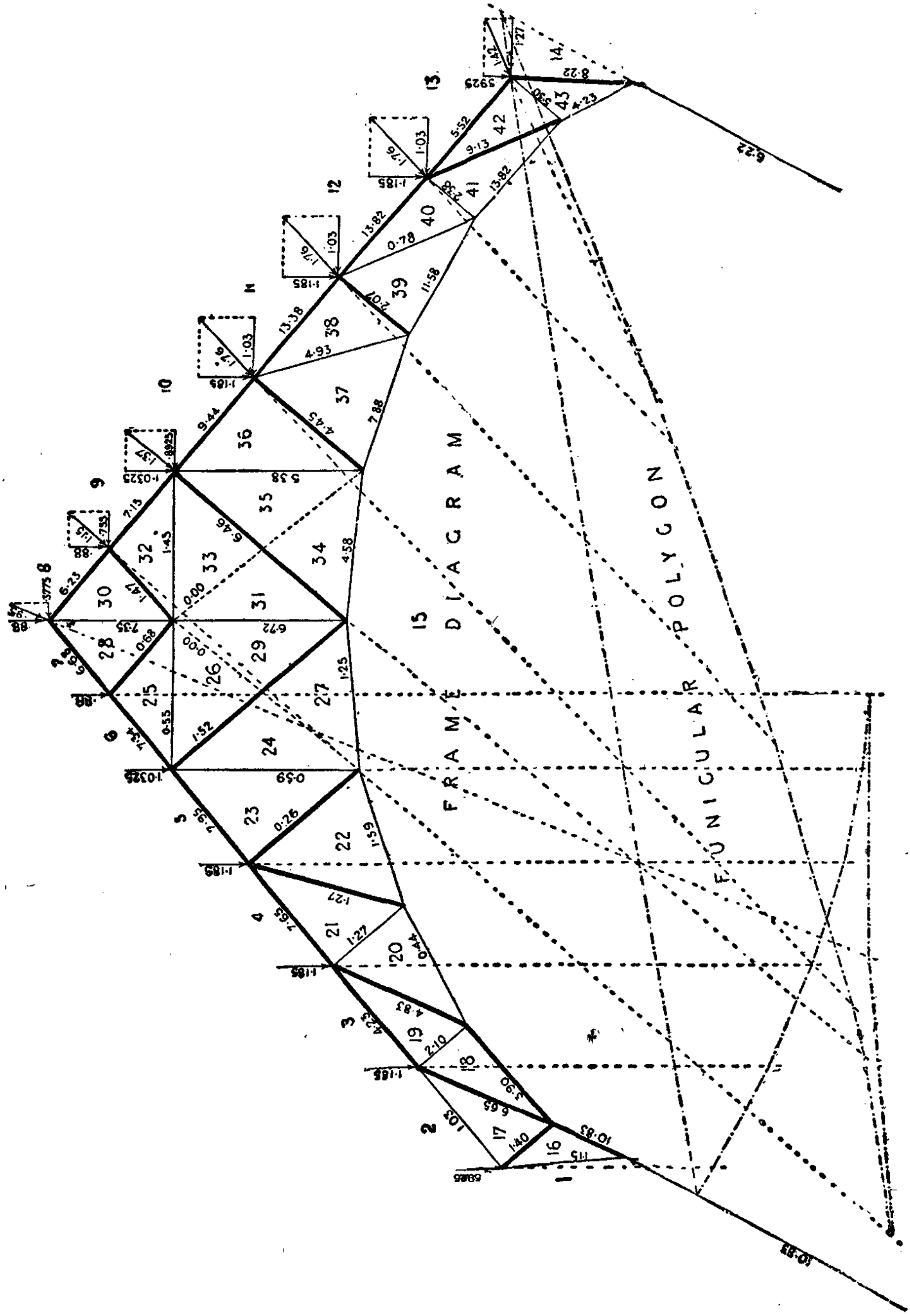
40
25
26
27
28
29
30
31
32

Figs. 1598 to 1601.—Diagrams showing Stresses in large Roof with Plate Webs.


FRAME DIAGRAM
FUNICULAR POLYGON
1
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3
4
5
6
8
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10
11
12
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16
17
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37
38
39
40
41
42
43
10·83
6·22
8·22
5·52
9·13
13·82
0·78
11·58
13·38
4·93
7·88
9·44
4·45
5·38
7·15
6·46
4·58
6·25
1·45
1·47
0·00
7·35
6·72
0·68
1·25
0·55
1·52
0·59
7·95
0·26
1·59
7·65
1·27
0·44
4·23
4·83
2·10
3·90
6·65
1·40
1·15
1·03
1·76
1·185
1·0325
·8925
·88
·755
·5925
·6885

Fig. 1602.—Frame Diagram of Roof Truss, 60-ft. Span.

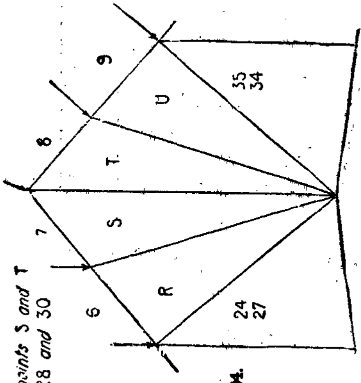
line 10—27, which will affect some of the stresses. To ensure accuracy, it should be worked out to a larger scale.

Diagram for Large Steel Truss.

The truss will be 80-ft. span, with a curved or segmental tie rod; the trusses 12 ft. apart, centre to centre, the roof covering of galvanised corrugated sheet iron 22 B.W.G., laid on felt, supported on battens; the latter secured to common rafters resting on purlins, and these purlins resting over the joints of the trusses. All weights, etc., are noted in Fig. 1596, and the accompanying illustration—Fig. 1597—gives the stress diagram.

Large Roof with Plate Webs.

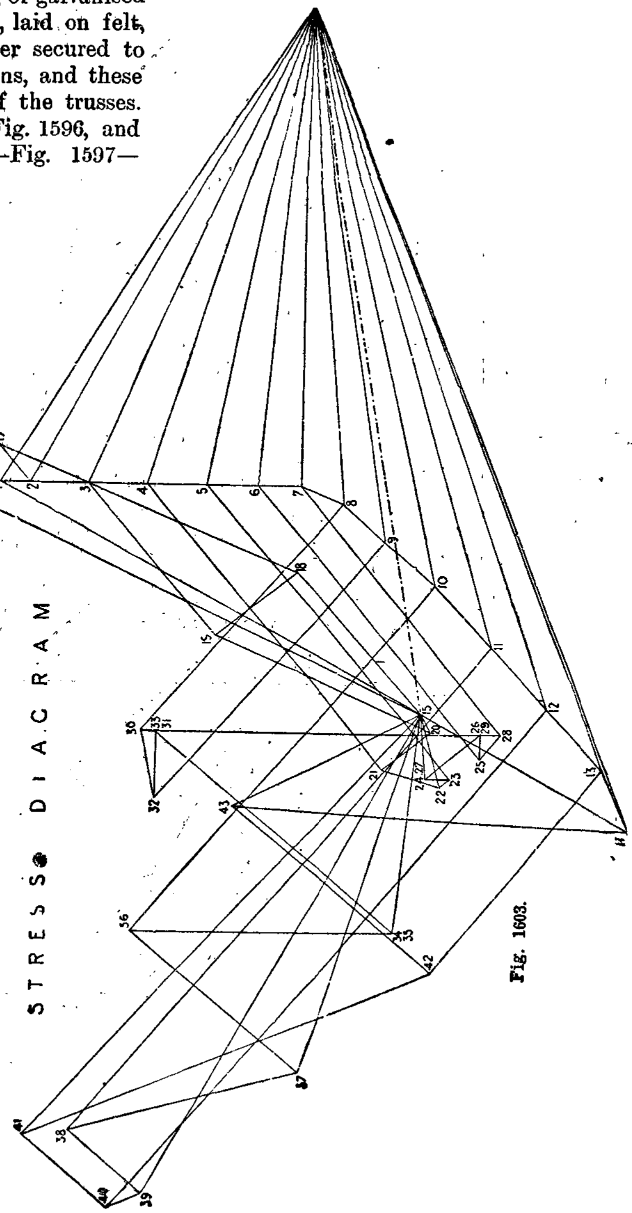
The stresses in a roof of this character are somewhat difficult to determine exactly. Any solid web in a girder or roof truss may be replaced by bars to enable a stress diagram to be drawn Fig. 1598 shows a sketch of the


9
8
7
6
U
T
S
R
35
34
24
27

Giving points S and T for 28 and 30

Fig. 1604.

STRESS DIAGRAM


17
1
2
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23
24
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38
39
40
41
42
43

Fig. 1603.

Figs. 1603 and 1604.—Stress Diagrams of Roof Truss, 60-ft. Span.

actual truss, Fig. 1599 the frame diagram as prepared for determining the stresses, Fig. 1600 the reciprocal stress diagram, and Fig. 1601 the nature of the stresses produced. Curved parts have to be indicated by straight lines parallel to the extremities of the curve, and allowance must be made for the transverse stress due to the outline not coinciding with the line of pull or push. As the tendency nowadays is to construct roofs and girders of forms not provided for in the text-books, it becomes increasingly necessary to study with care the principles of graphic statics.

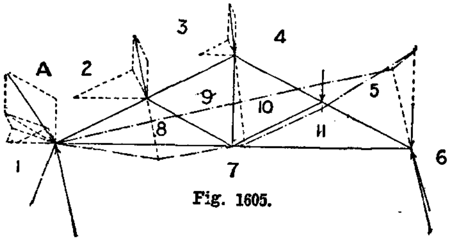


Fig. 1605.

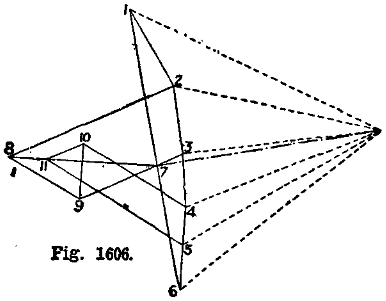


Fig. 1606.

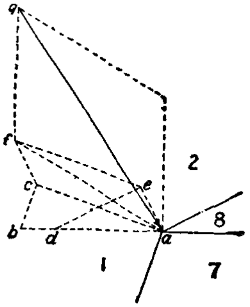


Fig. 1607

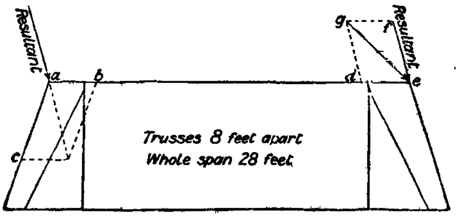


Fig. 1608.

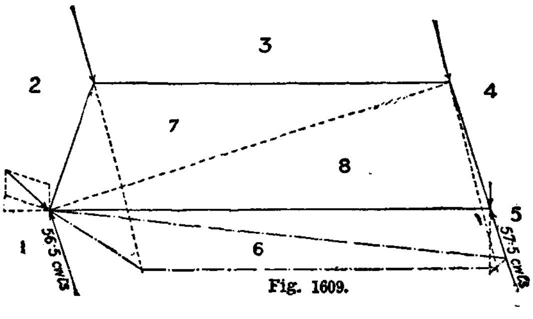


Fig. 1609.

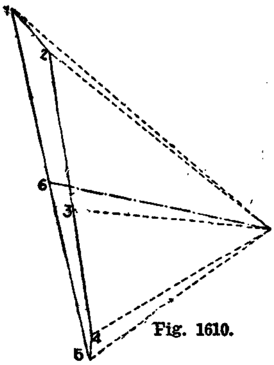


Fig. 1610.

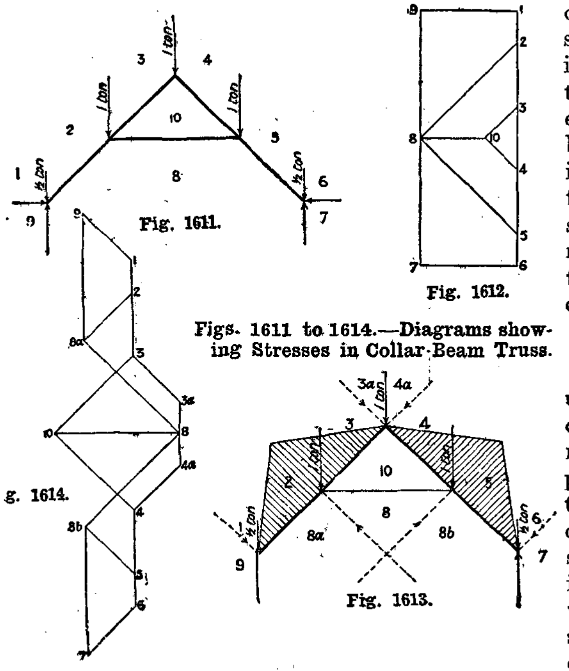
Figs. 1605 to 1610.—Diagrams showing Stresses in Mansard Roof.

Stresses in Roof Truss.

The truss of 60-ft. span shown in Fig. 1602 is rather a difficult and troublesome one to work out, owing to the number of members near the apex. The stress diagram (Fig. 1603) is not quite accurate, the closing line 30—32 not being quite parallel with the same line in the frame diagram, but the errors are probably very slight. In working this case the assumption is made that bars 24—27, 26—29, 31—33, and 34—35 are redundant and have no stress. Also a temporary rearrangement of members between 24 and 35 has to be made, as shown in Fig. 1604, in order to obtain points 28 and 30 on the stress diagram. The stresses found are marked against each member on the frame diagram. The wind cannot be considered to act on both sides of the roof at the same time unless it is taken vertically, but the maximum stresses on each side must be taken as acting similarly on both sides.

Stress Diagram for Mansard Truss.

Stress diagrams are required for the mansard roof truss loaded as shown, and with wind pressure of 50 lb. per square foot from the left. Taking the wind at 50 lb. per square foot, and assuming the trusses to be 8 ft. apart, the stress diagram is first found for the king-post truss as shown in Figs. 1605 and 1606. Fig. 1607 is an enlargement of A (Fig. 1605) to explain the mode of finding the forces. The half wind pressure on 1—7 is set off from *a* to *b*, then *a c* is drawn perpendicular to 1—7, and *b c* parallel to 1—7, to obtain the value of normal *a c*. Then *a d* is taken as the half wind pressure on 2—8, and the triangle completed to give the normal *a c*. This normal is then compounded with the previous one, giving the resultant *a f*, which is now compounded with the vertical load 1—2 to give the final resultant *a g*. The total effect of the king-post truss may be shown by the two resultants upon the lower truss as in Fig. 1608. Taking the left-hand resultant, it will be seen that it can be resolved into two directions, *a b* and *a c*; *a c* will merely compress the outer strut, and *a b* tends to push the whole frame over to the right. The queen-post and inner strut offer no resistance unless so secured as to permit of tension, and then a cross strain would be put upon the lower tie beam, showing that the whole arrangement is bad for resisting unsymmetrical stresses. The force *a b* may now be transferred to the right-hand side as *e d*, and compounded with the resultant *e f*, giving the final effort *e g*. The direction of this being outside the directions of both the struts, it is clear that, to prevent the truss from going over, the queen-post on the right-hand side should be secured top and bottom, so that the inner strut may be brought into play as well as the transverse strength of the tie-beam. The effect of the whole truss upon the walls may be found by putting a substituted member in the lower truss as in Fig. 1602, and making the stress diagram Fig. 1610. These remarks show in what way the principles of the truss might be improved, but it is generally a question of convenience that comes first, and the desire to utilise the chamber space in the roof.



Figs. 1611 to 1614.—Diagrams showing Stresses in Collar-Beam Truss.

Stress in a Collar-Beam Truss.

The stresses in a collar-beam truss depend not only upon the span, position of collar, pitch of roof, and nature of load, but also upon the nature of the support, whether perfectly rigid, partly yielding, or yielding so as to be unable to take any horizontal thrust. In the first case, the collar will be in compression, and the stresses slight; in the second, they will be indeterminate; and in the third, the collar will be in tension, the rafters under compression and at the same time under a very severe cross stress at the junction with collar, due to the leverage of the lower ends. In the accompanying diagrams an attempt has been made to compare the two extreme conditions for the same truss—Figs. 1611 and 1612 with firm buttresses so that the walls cannot spread, Figs. 1613 and 1614 for light walls incapable of resisting a thrust. The shaded portion shows the bending moments on the rafters, and the dotted force lines show the leverage effect of the ends of rafters.

Collar-Beam Roof with Wind at Side and Rigid Walls.

This problem has involved a somewhat long investigation, but its importance perhaps justifies the time spent on it. Fig. 1615 shows the frame diagram, with the external forces, which are assumed at 2 tons distributed for dead load, and 2 tons horizontally on one side for wind pressure. The particular difficulty is caused by the effect of wind pressure on one side of an imperfect truss. Fig. 1616 shows the upper portion of this truss, with the resistances obtained from the reciprocal diagram (Fig. 1617). Fig. 1618 shows the lower portion of the truss, with the forces resolved graphically, the parallelograms being reduced to half the scale of the forces. In Fig. 1619 the information obtained by Figs. 1616 and 1618 is brought into one view, and a reciprocal diagram (Fig. 1620) is made to suit it. These two diagrams are complete with the exception of the external force 2—2 *a*, which is non-existent, but is the true equilibrant of all the other forces, being equal and opposite to their resultant. In substitution, in Fig. 1621, half the amount must be transferred to the supports and combined with the reactions to produce the final resultants on the supports. In making this substitution, a bending-moment diagram must be added to the rafters as shown, where the horizontal force of ·525 ton on each side—that is, half the resultant, multiplied by the vertical height of 4 ft.—produces the bending moment of 2·1 ton-ft. at the joint with the collar-beam.

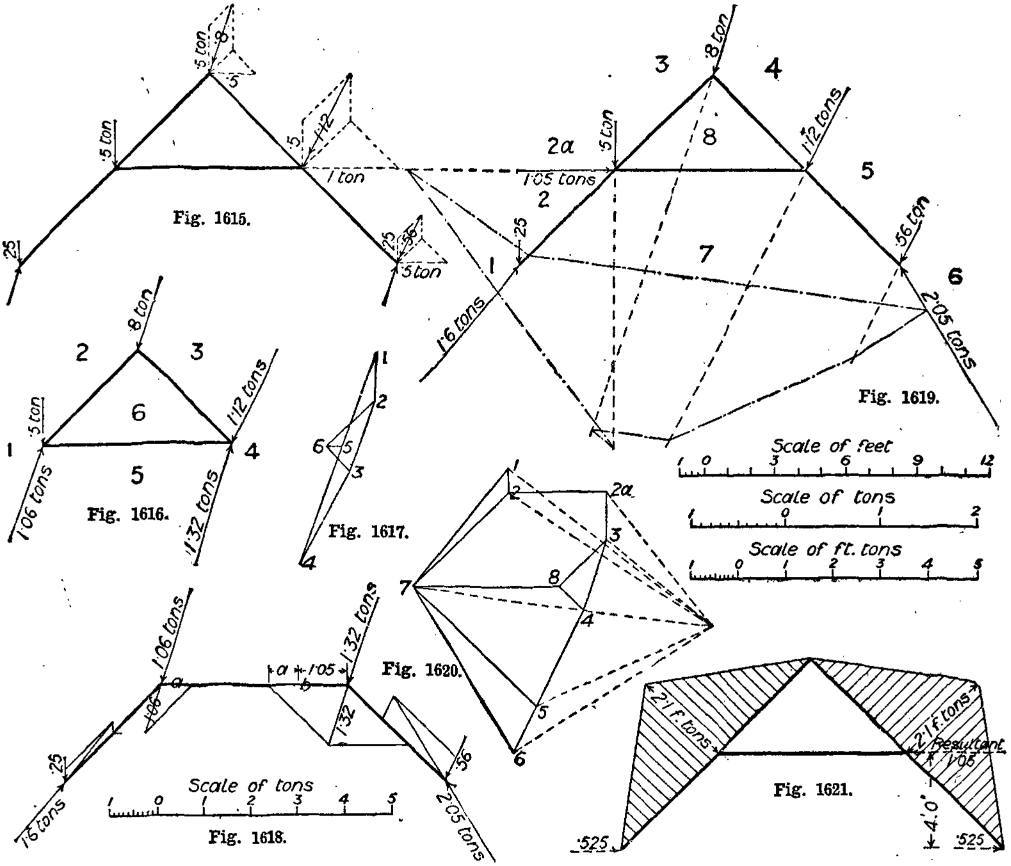


Fig. 1615.
·5 ton
·5 ton
1 ton
·25
·5 ton
Fig. 1616.
·8 ton
·5 ton
1·12 tons
1·06 tons
1·32 tons
Fig. 1617.
Fig. 1618.
1·06 tons
1·32 tons
1·05
Scale of tons
Fig. 1619.
·8 ton
1·12 tons
1·05 tons
·56 ton
2·05 tons
1·6 tons
Scale of feet
Scale of tons
Scale of ft. tons
Fig. 1620.
Fig. 1621.
2·1 ft. tons
Resultant 1·05
4'·0"
·525

Figs. 1615 to 1621.—Diagrams showing Stresses in Collar-Beam Roof with Wind at Side and Rigid Walls.

Collar-Beam Roof with Wind at Side and Walls not Rigid.

The determination of the stresses in a collar-beam truss has engaged the attention of many students and others for a long time past. Collar-beam trusses of one shape or another are used more than any other form for roofing churches, and are therefore of considerable importance. In the last case the walls were to be taken as rigid, by reason either of their thickness or of sufficient buttressing. The most important case, however, occurs when the wind blows on one side of the roof and the walls are not rigid. In Fig. 1622 is shown the frame diagram under the new conditions. The force lines for the dead loads are first added, then those for the wind, and the latter resolved into a normal and a sliding force; the normal being compounded with the vertical load to give the final forces as in Fig. 1623. The reactions are obtained by vectors Fig. 1624, and funicular polygon Fig. 1623, giving point 5. Then the reaction at A, multiplied by leverage *a*, gives the bending moment on the rafter. This, divided by the length of lower portion *b*, gives the virtual force *c*, acting at right angles to foot of rafter. A similar and equal force will be exerted at head of rafter; and a contrary force of the combined value will resist the other two, at junction with collar, half-way up the roof. A similar proceeding must be adopted for the other rafter; and the whole of the forces, virtual and actual, will then be as shown in Fig. 1625, from which the reciprocal diagram (Fig. 1626) may be constructed. The effect of the spreading tendency will be to cause bending moments on the rafters, as shown in Fig. 1627, the dotted forces in Fig. 1625 being equivalent.

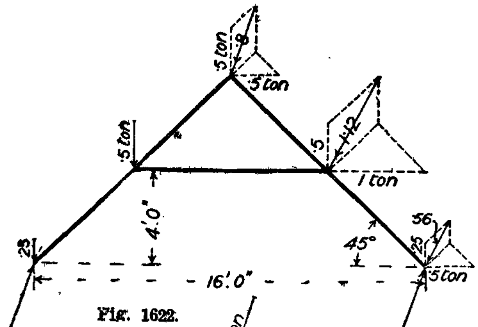

·5 ton
·5 ton
·5 ton
·56
1·12
1 ton
·56
4'·0"
45°
·5 ton
16'·0"

Fig. 1622.

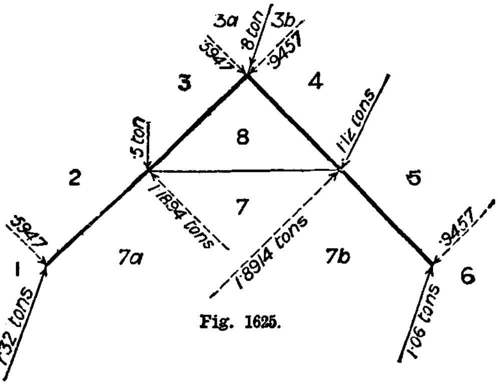

3a
·8 ton
3b
·3947
·9451
3
4
·5 ton
8
1½ tons
2
5
7
1·1894 tons
·3947
7a
1·8914 tons
7b
·9451
1
6
1·32 tons
1·06 tons

Fig. 1625.

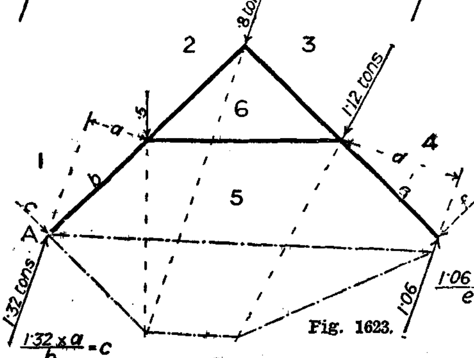
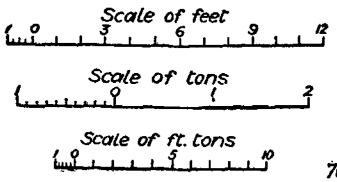

·8 ton
2
3
1·12 tons
6
·5
a
1
b
4
a
5
c
A
1·32 tons
1·06
1·06 x d / e = f
1·32 x a / b = c

Fig. 1623.


Scale of feet
0 3 6 9 12
Scale of tons
0 1 2
Scale of ft. tons
0 5 10

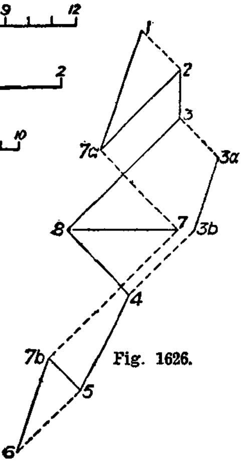

1
2
3
3a
7a
8
7
3b
4
7b
5
6

Fig. 1626.

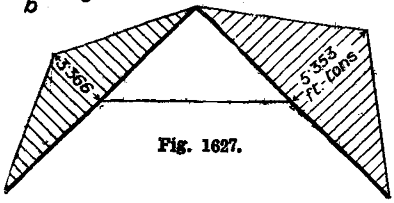

3·366
5·353 ft. tons

Fig. 1627.

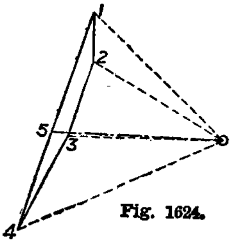


Fig. 1624.

Figs. 1622 to 1627.—Diagrams showing Stresses in Collar-Beam Roof with Wind at Side and Walls not Rigid.

STRENGTH OF BEAMS AND GIRDERS.

Reactions of Supports.

WHEN a beam carries a uniformly distributed load, or a central concentrated load, the reaction at each support is equal to half the load, but when the load is not symmetrically placed the reactions will be unequal. Fig. 1628 shows the elevation of a beam carrying a load. The proportion of the load transmitted to the supports A and B is found as follows:—

Load at A $= 3 \times \frac{10}{14} = 2\frac{1}{7}$

Load at B $= 3 \times \frac{4}{14} = \frac{6}{7}$

Total ... ... 3 tons.

The portion of the beam above the neutral layer (or, practically, the upper half of the beam) is under compression, the maximum stress being in the top fibres. The portion below the neutral layer is in tension, the maximum stress being in the bottom fibres. The stress is greatest in each case directly under the load. Fig. 1629 shows the elevation of a wooden beam 9 in. by 14 in., supported at both ends and loaded at two points. It is placed with the deepest side vertical so as to carry the loads with the least possible deflection, and at the supports A and B the reactions due to the loads, as well as the weight of the beam itself (taken at ½ cwt. per foot run),

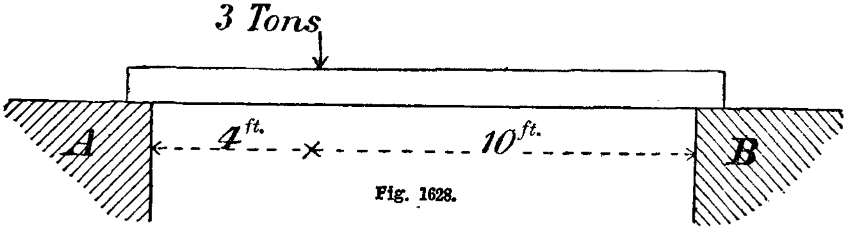

3 Tons
A
B
4 ft.
10 ft.
Fig. 1628.

Fig. 1628.—Elevation of Loaded Beam.

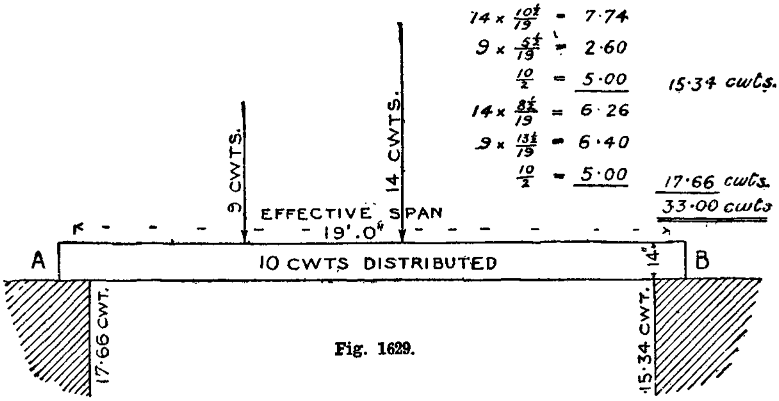

$14 \times \frac{10\frac{1}{2}}{19} = 7.74$
$9 \times \frac{5\frac{1}{2}}{19} = 2.60$
$\frac{10}{2} = 5.00$
15·34 cwts.
$14 \times \frac{8\frac{1}{2}}{19} = 6.26$
$9 \times \frac{13\frac{1}{2}}{19} = 6.40$
$\frac{10}{2} = 5.00$
17·66 cwts.
33·00 cwts
9 CWTS.
14 CWTS.
EFFECTIVE SPAN
19'.0"
A
B
10 CWTS DISTRIBUTED
14"
17·66 CWT.
15.34 CWT.
Fig. 1629.

Fig. 1629.—Beam Loaded at Two Points.

are marked as ascertained by the calculations shown.

Strength of Beam Loaded out of Centre.

Should it be desired to ascertain what weight a fir beam 13 in. by 7 in. will safely carry over a span of 11 ft., the weight being placed 7 ft. from one end, the beam should be sketched as shown in Fig. 1630; then it will be seen that the load on A will be $\frac{7}{11}$ W, and on B $\frac{4}{11}$ W. The bending moment will be greatest under the load, and equal to $\frac{7}{11}\text{W} \times 4$ or $\frac{4}{11}\text{W} \times 7$, both being equivalent to $\frac{28}{11}\text{W} = 2{\cdot}\dot{5}\dot{4}\ \text{W}$. If the load were central, the bending moment would be $\frac{1}{2}\text{W} \times \frac{11}{2} = \frac{11}{4}\text{W} = 2{\cdot}75\ \text{W}$. A fir beam, 13 in. by 7 in., with a central load on a span of 11 ft., will carry safely $\frac{1}{2}\frac{b\,d^2}{\text{L}} = \frac{7 \times 13^2}{11}$ $= 53{\cdot}77$ cwt. The safe load will vary inversely as the bending moment produced by its position; therefore we have $53{\cdot}77 \times \frac{2{\cdot}75}{2{\cdot}\dot{5}\dot{4}}\text{W} =$ $53{\cdot}77 \times 1{\cdot}08 = 58$ cwt. A shorter and more direct way, but not so easily reasoned out, is $\frac{1}{2}\frac{b\,d^2}{\text{L}} \times \frac{(\frac{1}{2}\text{L})^2}{x \times y} = \frac{1}{2}\frac{7 \times 13^2}{11} \times \frac{5{\cdot}5^2}{28} =$ 58 cwt.

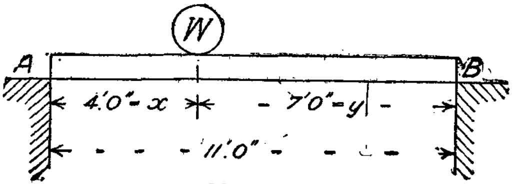


Fig. 1630.

Fig. 1630.—Elevation of Loaded Beam.

Stresses in Loaded Beam.

When a beam is fixed at both ends and loaded as shown in Fig. 1631, the nature of the stresses set up in the beam at A, B, and C will be as follows: At A and B the top of the beam will be in tension and the bottom in compression, these stresses reducing gradually towards the middle of the depth, where they disappear at the neutral layer. At C the stresses are reversed; in other respects the same remarks apply. The change from the conditions at A and B to those at C occurs gradually, the reversal taking place at the points of contrary flexure one-fourth of the span from each end. There is also a shearing stress throughout the beam, which is equal to half the load. At C the load produces deflection of the beam, so that the upper part is concave, while it becomes convex at A and B. The distribution of the stresses is shown by thick lines for compression and thin for tension, *d* and *e* being the points of contrary flexure.

Moment of Inertia of Beam.

The moment of inertia I at a cross-section of a beam is the summation of the areas of the individual fibres multiplied by the squares of their distances from the neutral axis. In a rectangular beam it will stand as

$$\text{I} = \Sigma\, a\, y^2 = \frac{b\,d^3}{12},$$

and the modulus of section Z, generally used in calculations of strength, is this value divided by the distance of the line of maximum stress from the neutral axis, which is half the depth, or

$$\text{Z} = \frac{b\,d^3}{12} \div \frac{d}{2} = \frac{b\,d^2}{6}.$$

Moment of Resistance of Beam.

The moment of resistance R is the product of the modulus of section Z into the modulus of rupture C. The modulus of rupture is assumed to be eighteen times the load required to break a bar of 1 sq. in. section, supported on two points 1 ft. apart, and loaded in the centre. C = for fir 5,000 to 10,000, for oak 10,000 to 13,000. For example, take a fir beam, 9 in. by 6 in. and 10-ft. span, load distributed, and find the breaking weight.

Effort = resistance

Bending moment M = moment of resistance R

$$\frac{\text{W}\,l}{k} = \text{Z}\,\text{C}$$

$$\therefore \text{W} = \text{Z}\,\text{C}\frac{k}{l}.$$

From the notes above, $\text{Z} = \frac{1}{6}\,b\,d^2$, $\text{C} = 7{,}500$, k = coefficient of reaction, according to method of loading and supporting, which in this case will equal 8; l = clear span in inches. Then

$$\text{W} = \text{Z}\,\text{C}\frac{k}{l} = \frac{b\,d^2}{6} \times 7{,}500 \times \frac{8}{l}$$

$$= \frac{6 \times 9 \times 9}{6} \times 7{,}500 \times \frac{8}{10 \times 12} = 40{,}500\ \text{lb.}$$

= 18 tons, or, say, 3 tons safe load distributed.

The expression for moment of resistance will thus be

$$R = Z C = \frac{c\,b\,d^2}{6},$$

and C in this formula is commonly understood to be the same as lb. per square inch maximum stress, but it is really the coefficient of bending strength, and is usually in excess of the actual tensile strength of the material.

Modulus of Rupture.

A misapprehension of the relationship between the maximum fibre stress and the modulus of rupture is very general. The common equation for a rectangular beam is $M = C\frac{bd^2}{6}$, where M = bending moment, C = coefficient of transverse strength, which is generally supposed to be the same as maximum fibre stress, but is not so. This is a very important matter, and it is well to know what authorities say about it. Humber's "Handy Book for the Calculation of Strains in Girders, and Similar Structures," says: "Modulus of Rupture.—The theoretical value of C is the resistance of the material to direct compression or tension, but it is found from experiments on cross breaking that this value is not sufficiently high. Amongst the reasons that have been assigned for this are: (1) That in addition to the resistances of the particles of the beam to a direct strain, there is another resistance arising from the lateral adhesion of the fibres to each other, termed the 'Resistance of flexure' (see Barlow on the 'Strength of Materials,' 6th edition); and (2) that in most metallic beams (especially when cast) the outer skin, which is strained more than any other part of the section, is very much stronger (from many well-known causes) than the average section; whereas if the direct tensile or compressive resistance of the same beam, in the direction of its length, were being experimentally ascertained, it would be the average section at least, and perhaps the centre (weaker) portion especially, from which the strength would be determined. However, there is evidently a necessity to employ a higher value than that for the direct resistance; and Professor Rankine has adopted a modulus of rupture which is eighteen times the load required to break a bar of 1 sq. in. section, supported on two points 1 ft. apart, and loaded in the middle between the supports." Professor Fidler's "Bridge Construction" says, with regard to the common theory: "The simplicity of this theory would be very satisfactory if it could be regarded as a true and complete statement of the facts; for nothing could be easier than to calculate by the last formula the weight required to produce any given tensile stress; and if we know the ultimate tensile strength of the material, it would seem that we ought to be able, by this means, to find exactly the load that will break the beam. But if we take a rectangular beam of cast iron, and put the calculated breaking load upon it, the beam will show no symptoms of tearing at the stretched fibres, and no inclination to yield in any way; and, as a matter of fact, it will not break until we have increased the load to about 2¼ times the amount calculated." R. H. Cousins says: "Experiments have shown that the compressive and tensile strength do not possess equal values as factors in determining the transverse load that a beam will bear, and that the influence of the tensile strength predominates." The whole subject is one of considerable difficulty, although from want of sufficient knowledge it is generally considered extremely simple.

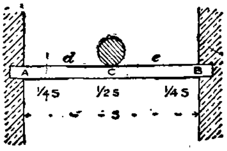


Fig. 1631.

Fig. 1631.—Elevation of Loaded Beam.

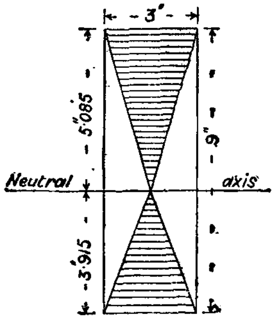
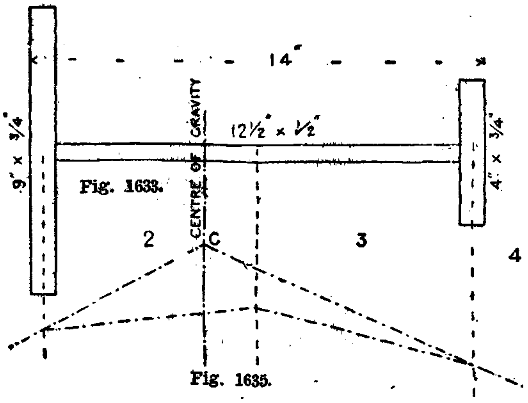


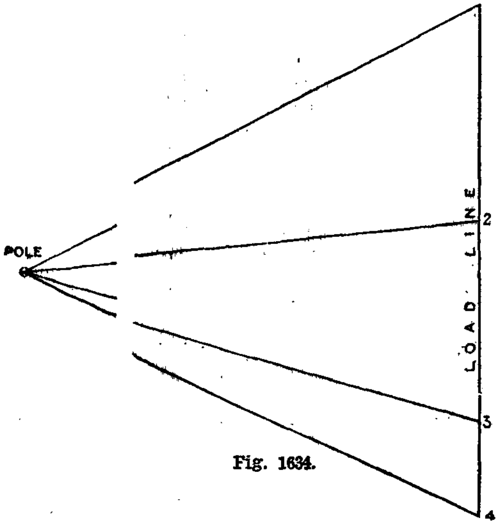
Fig. 1632.

Fig. 1632.—Determining Resisting Couple of Beam.

Resisting Couple of a Beam.

Assuming that the maximum strength of a beam under transverse loading is made up of its resistance to compression and tension, an attempt may be made to equate the resisting couple as follows: Beam, 9 in. depth by 3 in. breadth. Compressive strength, 3 tons per sq. in. Tensile strength, 5 tons per sq. in. Distance of neutral axis from lower edge at instant of rupture $= \frac{cd - d\sqrt{tc}}{c - t} = \frac{27 - 9\sqrt{15}}{-2} = 3{\cdot}915$ in., as illustrated in Fig. 1632. The stresses being assumed to vary with the distance from neutral axis, there will be a resistance of $\frac{3 \times 5{\cdot}085}{2} \times 3 = 22{\cdot}825$ tons acting with a leverage of $\frac{2}{3}$ (5·085) = 3·39 in., or a moment of 22·825 × 3·39 = 77·57 ton-ins. on the compression side, and $\frac{3 \times 3{\cdot}915}{2} \times 5 = 29{\cdot}3625$ tons acting with a leverage of $\frac{2}{3}$ (3·915) = 2·61 in., or a moment of 76·64 ton-ins. on the tension side, making a total moment of 154·21 ton-ins., the equivalent of what is called "the greatest resisting couple." If exact decimal places had been taken, the two moments would, of course, have been equal. It may be remarked that, assuming the extension and compression to be equal within the limit of elasticity of the material, the neutral axis will pass through the centre of gravity of the section; but there is reason to suppose that this axis shifts towards the stronger side as this limit is passed, otherwise the stress must be equal in the top and bottom fibres, and the strength of the beam would be measured by the weaker stress only.





Figs. 1633 to 1635.—Obtaining Centre of Gravity of Cast-Iron Girder.

Centre of Gravity of Cast-Iron Girder.

The centre of gravity of a cast-iron girder, or other girder of irregular section, may be found by the funicular polygon as follows:—Draw the section of girder as in Fig. 1633, treating the areas as if they were loads, drawing the force lines separating spaces 1, 2, 3, 4, and making the distances on the load line in Fig. 1634 correspond to scale with the areas of Fig. 1633. Select any point for a pole as O (Fig. 1634), and draw lines to the points on the load line. Across the force lines from Fig. 1633, and parallel to the lines of Fig. 1634, draw the lines of the funicular polygon (Fig. 1635), then the lines across spaces 1 and 4 meet at C, through which passes the required centre of gravity line. Wrought-iron girders being generally symmetrical, the centre of gravity is at the middle of the depth on the centre line. Cast-iron girders and unsymmetrical sections may be dealt with as follows:—The area of each part multiplied by the distance of its centre of gravity from a base line and divided by the total area will give the height of the mean centre of gravity above the base, thus $\frac{\text{A}a + \text{B}b + \text{C}c}{\text{A} + \text{B} + \text{C}} = x$ (see Fig. 1636). If the web be taper, its centre of gravity is most easily found in the same way as that of a retaining wall, and is shown in Fig. 1637. Add the top width on each side of the bottom, and the bottom width on each side of the top, draw the diagonals, and the intersection will be the centre of gravity.

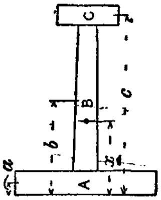


Fig. 1636.

Fig. 1636.—Obtaining Centre of Gravity of Cast-Iron Girder.

Moment of Inertia of Girder.

The moment of inertia may be found as shown in Fig. 1638, where *c g* is the mean centre of gravity. Lines are drawn from the outer angles of the top and bottom flange to this point to give the inertia area of each flange. Then the web ends are projected on to the outside of the flanges, and lines drawn to the centre of gravity to give the inertia areas of the web. The moment of inertia is the sum of the area of each shaded part multiplied by the distance of its centre of gravity from the mean centre of gravity, multiplied by the distance from the mean centre of gravity to the extreme fibre on that side.

Definitions of Moment of Inertia and Radius of Gyration.

A concise definition of the moment of inertia of a section is, "the sum of the products of the resistances of all the parts into the squares of their distances from the neutral axis," the usual formula being $I = \Sigma\, a\, y^2$, I being moment of inertia, Σ signifying summation, *a* area of any small portions of section, *y* distance of the centre of gravity of the portion from the neutral axis. For the radius of gyration, if the area of a section be assumed to have a small but appreciable thickness, "the radius of gyration is the distance from the assumed axis of revolution (usually either the neutral axis or one edge) to that point at which, if the whole mass were collected, the same angular velocity would be generated (in vacuo) in the same time by the same force as when acting upon the area itself"; the usual formula being $r = \sqrt{\frac{I}{A}}$, or $I = Ar^2$, I being moment of inertia as before, A area of section, and *r* radius of gyration.

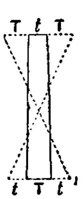


Fig. 1637.

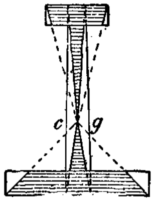


Fig. 1638.

Fig. 1637.—Finding Centre of Gravity of Girder with Taper Web. Fig. 1638.—Finding Moment of Inertia of Girder.

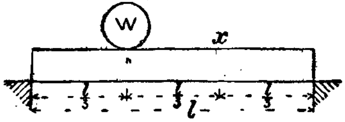


Fig. 1639.

Fig. 1639.—Elevation of Loaded Beam.

Bending Moments.

The bending moments on a loaded beam may be shown very simply by graphic methods. When the beam is supported at the ends and carries a concentrated load in the centre, the bending moments will be as ordinates to a triangle whose height is $\frac{WL}{4}$, and when the load is uniformly distributed the bending moments will be as ordinates to a parabola whose height is $\frac{WL}{8}$.

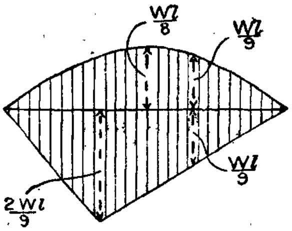


Fig. 1640.

Fig. 1640.—Stress Diagram for Loaded Beam.

Bending Moments for Combined Loads.

A beam, of weight W^1 (Fig. 1639), is supported at its ends in a horizontal position, and bears a load W at a point of trisection. The bending moment at the other point of trisection will be found as follows:—

Bending moment at x due to load W=

$$\frac{W \times \frac{l}{3}}{l} \times \frac{l}{3} = \frac{W}{1} \times \frac{l}{3} \times \frac{1}{l} \times \frac{l}{3} = \frac{W\,l}{9}.$$

Bending moment at x due to weight of beam W^1 =

$$\frac{W^1}{2} \times \frac{l}{3} - \frac{W^1}{3} \times \frac{l}{3} \times \frac{1}{2} = \frac{W^1\,l}{6} - \frac{W^1\,l}{18} = \frac{W^1\,l}{9}$$

Total bending moment at x = $\frac{W\,l}{9} + \frac{W^1\,l}{9}$.

The form of the complete stress diagram will be as shown in Fig. 1640.

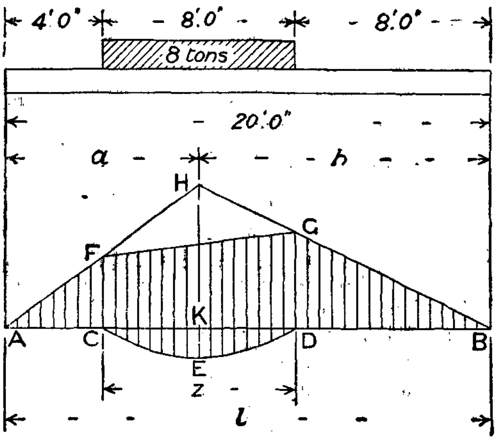

4'0"
8'0"
8'0"
8 tons
20'0"
a
b
H
G
F
K
A
C
D
B
E
z
l

Fig. 1641.

Fig. 1641.—Bending Moment Diagram for Partial Loading.

Bending Moment Diagram for Partial Loading.

To show the method of producing the bending moment diagram for a given case, draw the outline of the girder and load as shown in Fig. 1641, then let A B = the whole span, C D part occupied by load, K centre of load dividing span into $a + b$, w = load per foot run, z = length occupied by load, then K H = $\frac{w\,z\,a\,b}{l}$, points F and G cut off the top of triangle at each end of load, K E = $\frac{w\,z^2}{8}$ with parabola for outline, and the whole of the moments are given by the shaded portion. Let w = 1 ton, z = 8 ft., K H = $\frac{w\,z\,a\,b}{l} = \frac{1 \times 8 \times 8 \times 12}{20}$ = 38·4 ton-feet, K E = $\frac{1 \times 8^2}{8}$ = 8 ton-feet. The bending moment cannot be found by simply taking the load as collected at its centre of gravity, the small parabola not being the same area as the piece of triangle cut away.

Strength of Rolled Joists Cased in Concrete.

When the joists form more than say 10 per cent. of the whole mass, they must be calculated independently. As an example, assume the general diagram of loading to be as shown in

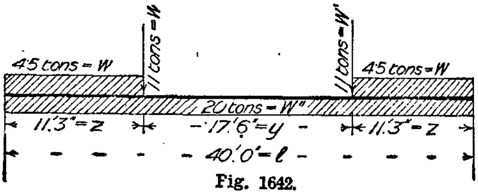
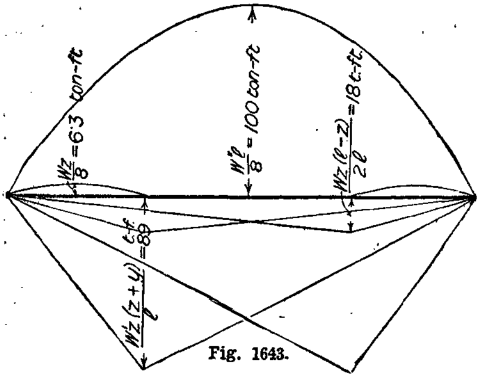
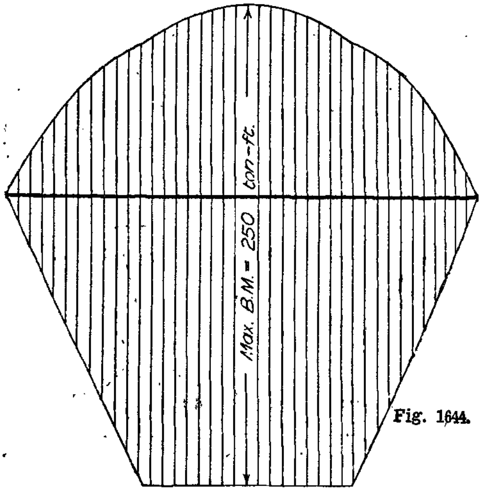

4·5 tons = W
11 tons = W'
11 tons = W'
4·5 tons = W
20 tons = W''
11'3" = z
17'6" = y
11'3" = z
40'0" = l

Fig. 1642.


Wz/8 = 6·3 ton-ft
W''l/8 = 100 ton-ft
Wz(l-z)/2l = 18 t-ft
W'z(z+y)/l = 89 t-f
Fig. 1643.


Max. B.M. = 250 ton-ft.
Fig. 1644.

Figs. 1642 to 1644.—Finding Strength of Rolled Joists Cased in Concrete.

Fig. 1642, the graphic diagram of bending moments will be as shown in Fig. 1643, and the final diagram of collected bending moments as shown in Fig. 1644. The maximum bending moment is found to be 250 ton-feet on the span of 40 ft. This is without allowing anything for either the weight or the strength of the concrete that surrounds the rolled joists; possibly the concrete as used may be of sufficient value to carry its own weight, but not more. A 20-in. by $7\frac{1}{2}$-in. by 89-lb. joist on a 40-ft. span at 8 tons per square inch will carry $\frac{878}{40} = 21{\cdot}9$, say 22 tons distributed, producing a bending moment of $\frac{\text{W L}}{8} = \frac{22 \times 40}{8} = 110$ ton-feet, so that two of these joists will hardly be sufficient, and 18-in. by $\frac{5}{8}$-in. plates top and bottom will be required, making Dorman, Long and Co.'s G1 C2 260-lb. compound girder. Two rolled joists of equivalent strength are more economical than a compound girder.

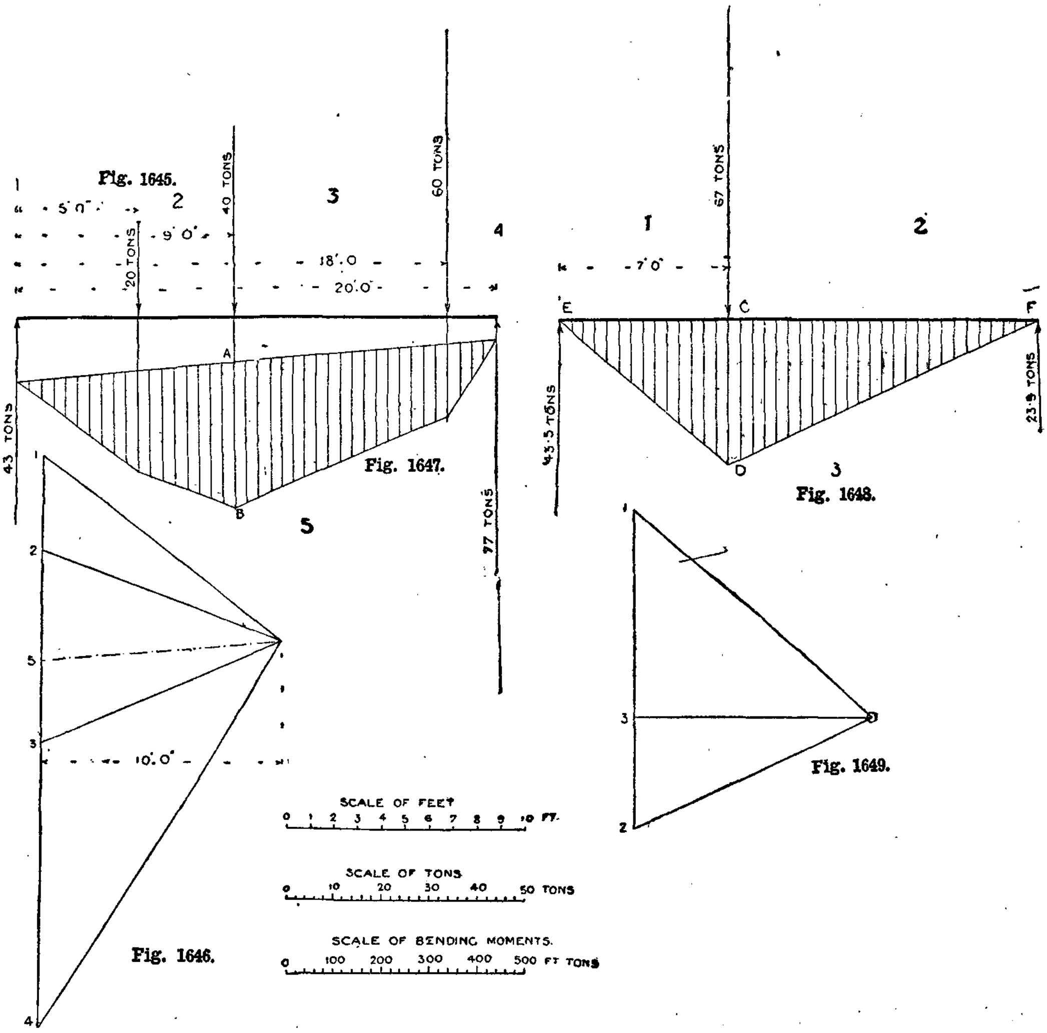


Fig. 1645.

Fig. 1646.

Fig. 1647.

Fig. 1648.

Fig. 1649.
1
5' 0"
2
9' 0"
20 TONS
40 TONS
3
60 TONS
4
18'.0
20'.0
A
43 TONS
97 TONS
B
5
1
2
5
3
4
10'.0"
67 TONS
1
2
7' 0"
E
C
F
43.5 TONS
23.5 TONS
D
3
1
3
2
SCALE OF FEET
0 1 2 3 4 5 6 7 8 9 10 FT.
SCALE OF TONS
0 10 20 30 40 50 TONS
SCALE OF BENDING MOMENTS.
0 100 200 300 400 500 FT TONS

Figs. 1645 to 1649.—Diagrams showing Equivalent Loads on Beams.

Equivalent Loads on Beam.

Take the case of a beam of 20-ft. span carrying three loads, 20 tons, 40 tons, and 60 tons, at 5 ft., 9 ft., and 12 ft. from one end. These loads are to be removed, and a single load placed 7 ft. from one end. Determine the single load that shall produce the same maximum bending moment as the combined effect of the three loads. Then Fig. 1645 shows the beam with loads to scale; Fig. 1646 the polar diagram; Fig. 1647 the shaded portion, being the funicular polygon or bending moment diagram. From this it is seen that the maximum bending moment is under the load of 40 tons, and its amount is the length A B. Then to find the equivalent single load at 7 ft. from the end, draw a fresh frame diagram (Fig. 1648), and under the point of application of the load drop the perpendicular C D equal to the maximum bending moment A B. Join E D, D F. Then E D F is the bending moment diagram for the beam as now loaded. Take any pole O (Fig. 1649), draw an indefinite vertical load line at the same distance from the pole as in the first case, and from O draw O 1 parallel to the line E D, also O 2 parallel to D F. The length of the load line 1—2 thus determined will be the single load required, which scales 67 tons.

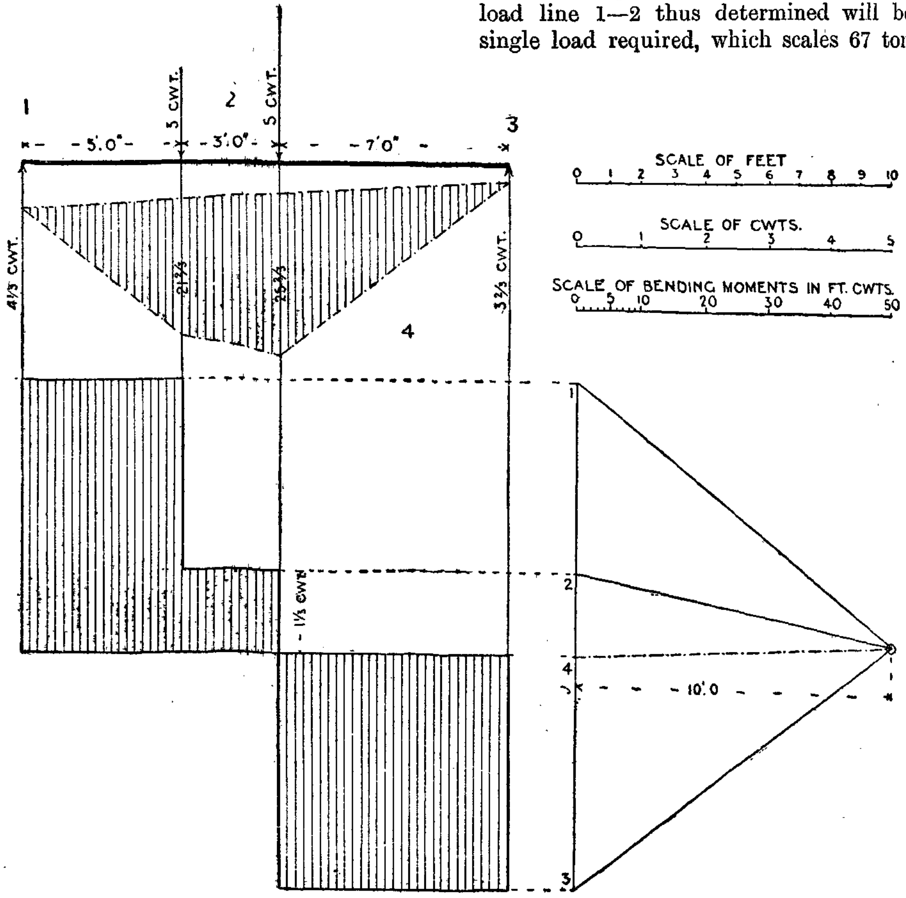
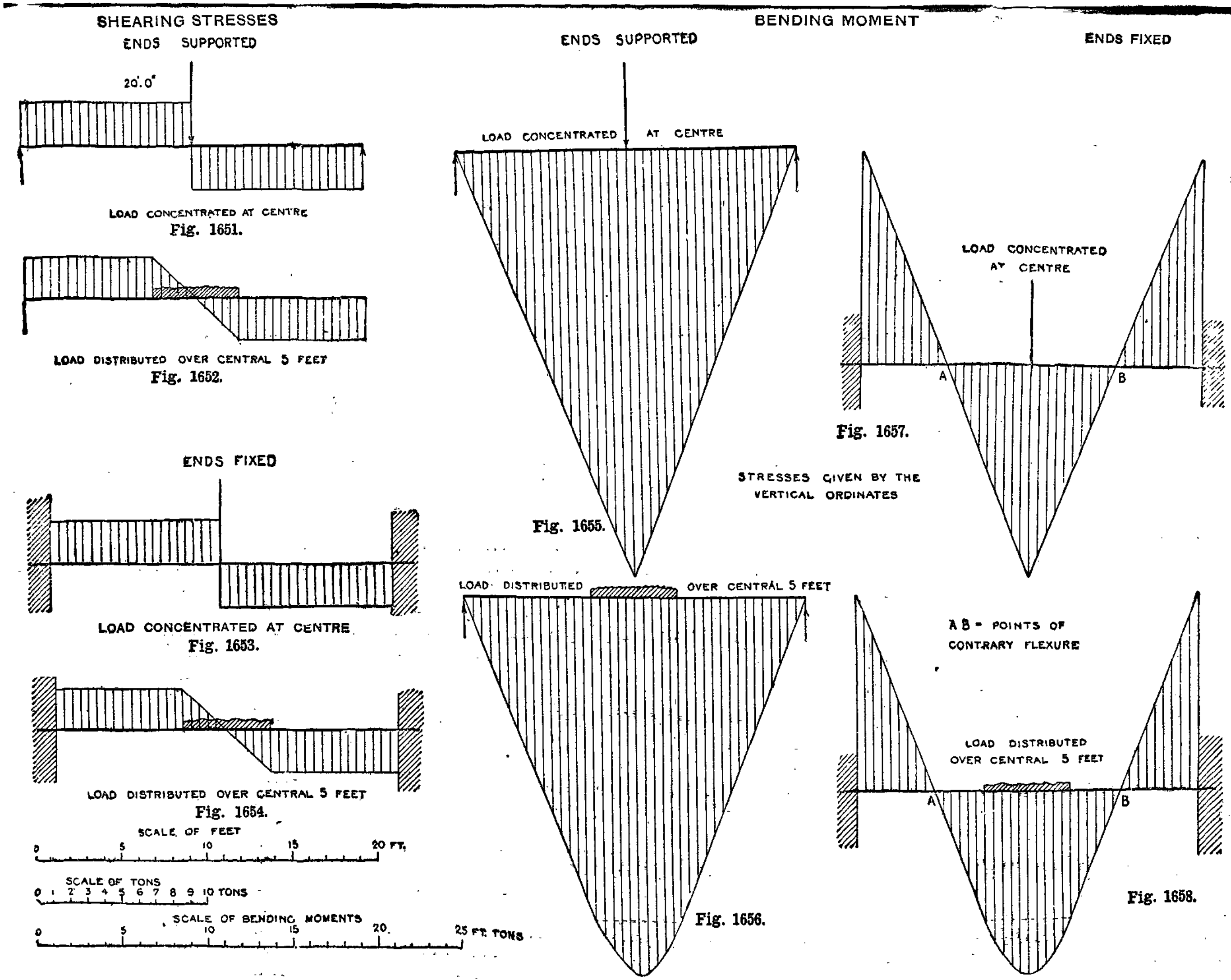


Fig. 1650.—Combined Bending and Shearing Diagrams.

Combined Bending and Shearing Diagrams.

A neat method of combining bending moment and shearing stress diagrams is shown in Fig. 1650, where a beam supported at both ends, over a 15-ft. span, carries a load of 3 cwt. at a point 5 ft., and 5 cwt. at 8 ft. from one of the supports. A line of loads 1—2, 2—3, is drawn as for a reciprocal diagram, a pole O taken at a convenient distance, say 10 ft., and


SHEARING STRESSES
ENDS SUPPORTED
20.0"
LOAD CONCENTRATED AT CENTRE
Fig. 1651.
LOAD DISTRIBUTED OVER CENTRAL 5 FEET
Fig. 1652.
ENDS FIXED
LOAD CONCENTRATED AT CENTRE
Fig. 1653.
LOAD DISTRIBUTED OVER CENTRAL 5 FEET
Fig. 1654.
SCALE OF FEET
0 5 10 15 20 FT.
SCALE OF TONS
0 1 2 3 4 5 6 7 8 9 10 TONS
SCALE OF BENDING MOMENTS
0 5 10 15 20 25 FT. TONS
BENDING MOMENT
ENDS SUPPORTED
LOAD CONCENTRATED AT CENTRE
Fig. 1655.
LOAD DISTRIBUTED OVER CENTRAL 5 FEET
Fig. 1656.
STRESSES GIVEN BY THE VERTICAL ORDINATES
ENDS FIXED
LOAD CONCENTRATED AT CENTRE
A
B
Fig. 1657.
A B = POINTS OF CONTRARY FLEXURE
LOAD DISTRIBUTED OVER CENTRAL 5 FEET
A
B
Fig. 1658.

Figs. 1651 to 1658.—Shearing Stresses and Bending Moments for Beams under Various Conditions.

vectors drawn. From these a funicular polygon is constructed below the girder, and the closing line gives point 4 on the line of loads, from which the reactions 3—4 and 4—1 are obtained. The shear stress diagram is then found by projecting verticals from the force lines on the girder and horizontals from the points on the polar diagram.

Comparison of Supported and Fixed Beams.

A beam over a 20-ft. span carries 5 tons at the centre. It is required to be shown by diagrams to a scale of 5 ft. and 5 tons to the

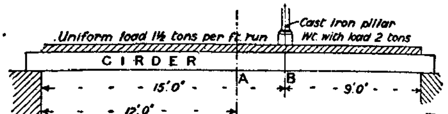

Uniform load 1¼ tons per ft. run
Cast iron pillar
Wt. with load 2 tons
GIRDER
A
B
15'0"
12'0"
9'0"

Fig. 1659.

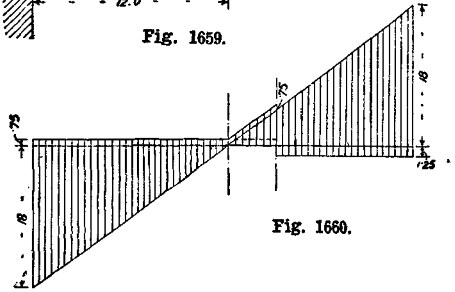


Fig. 1660.

Figs. 1659 to 1660.—Shear Stress Diagram for Girder supporting Column.

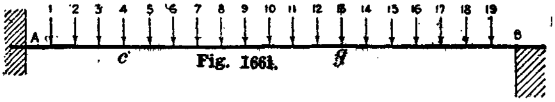

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
A
B
c
g

Fig. 1661.

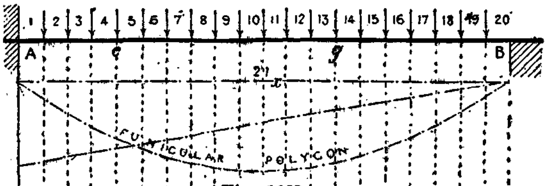

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
A
B
c
g
21
x
FUNICULAR POLYGON

Fig. 1662.

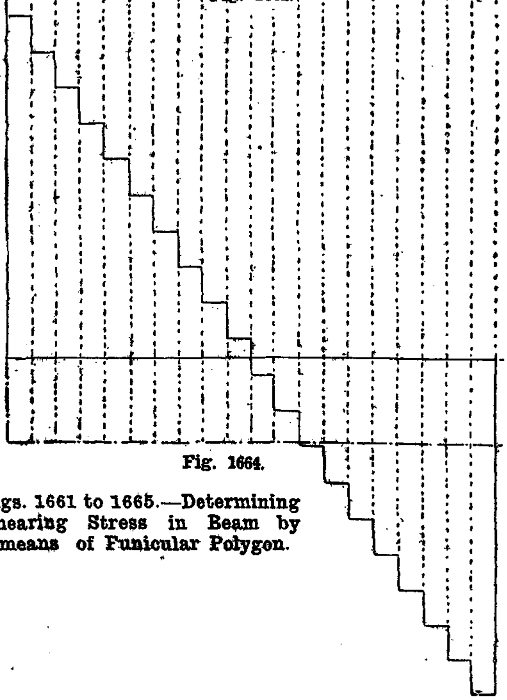


Fig. 1664.

Figs. 1661 to 1665.—Determining Shearing Stress in Beam by means of Funicular Polygon.

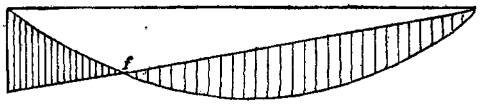


Fig. 1665.

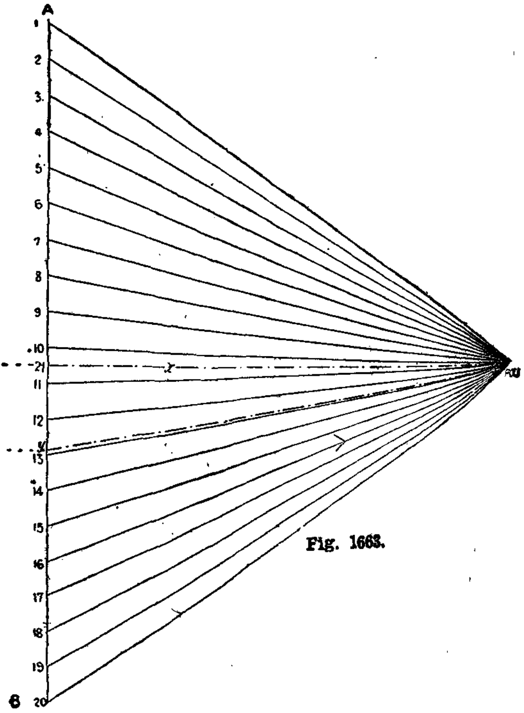

A
1
2
3
4
5
6
7
8
9
10
21
11
12
y
13
14
15
16
17
18
19
20
B
x

Fig. 1663.

inch, the distribution of the shearing stress and bending moments : (*a*) when the beam is supported at the ends ; (*b*) when the ends of the beam are fixed. Also taking the 5 tons as distributed over the central 5 ft. of the beam, show how the diagrams under (*a*) and (*b*) would be modified. Figs. 1651 to 1658 show the shearing stresses and bending moments for the beams under the various conditions given.

Shear Stress Diagram for Girder supporting Column.

Fig. 1659 shows the girder submitted, and Fig. 1660 the shear stress diagram, obtained as follows : For the distributed load of 1½ tons per foot run, the reaction at each end will be $\frac{4 \times 1\frac{1}{2}}{2}$ = 18 tons, which will also be the maximum shear stress due to this load. Set this off in Fig. 1660 below the base line of girder on left-hand side, and above on right-hand side, and join the points ; the ordinates of stress due to this load throughout the girder will then be as shown by the vertical lines of the two triangles. For the concentrated load the reaction on the left will be $\frac{2 \times 9}{24}$ = ·75 ton, and on the right $\frac{2 \times 15}{24}$ = 1·25 tons, together making up the whole load on column of 2 tons. Set off the values at each end and draw horizontal lines to the position of column, as shown, then the combined vertical ordinates give the combined shearing stresses throughout the girder. At centre they will equal ·75 ton, under the column = 5·75 tons, at left-hand support 18·75 tons, and at right-hand support 19·25 tons.

Shearing Stress in Beam Determined by Means of the Funicular Polygon.

A B (Fig. 1661) represents a horizontal beam 20 ft. in length, loaded at the points 1, 2, 3...... 19 ; the load at each of these is 1 ton. (*a*) The beam supposed resting freely on two supports at A and B. Determine by means of the funicular polygon the shearing stress (in tons) and the bending moment (in ton-feet) at the points *c* (4) and *g* (13). (*b*) The beam supposed built in firmly at A and resting freely on a support B. Assuming, what actually occurs in this case, that three-eighths of the total of all the loads supported by the beam are borne by the support at B, show how the measurements determined by the funicular polygon in case (*a*) can be modified by an additional line so as to obtain the stresses in this case. Determine the bending moment (in ton-feet) at A and *g* (13).

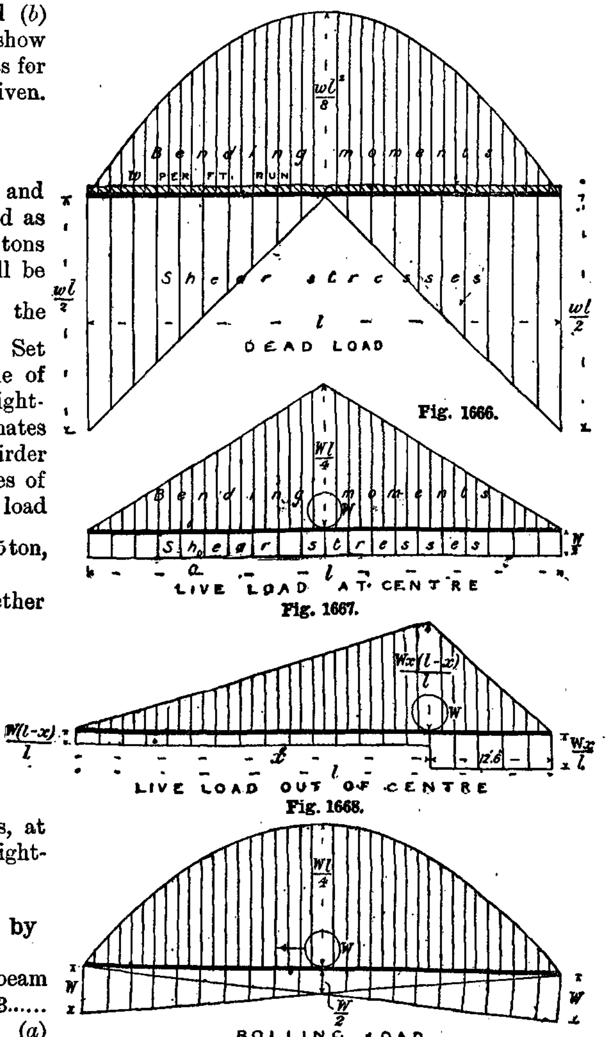


Fig. 1666.

Fig. 1667.

Fig. 1668.

Fig. 1669.

Figs. 1666 to 1669.—Stress Diagrams for Distributed, Concentrated, and Rolling Loads.

Scale of lengths, 4 ft. to the inch ; scale of loads, 1 ton to ⅜ in. ; employ for polar distance, 20 ft. Case *a :* Draw elevation of beam to required scale, as in Fig. 1661, and add space numbers according to Bow's notation shown in Fig. 1662. Draw line of loads, mark pole and draw vectors, as in Fig. 1663. By parallel lines from Fig. 1663, construct the funicular polygon shown on Fig. 1662; join the extremities to obtain line x and transfer it to Fig. 1663 to divide the load into the reaction amounts at A and B, which in this case are equal. Then construct the shear stress diagram by projection from Figs. 1662 and 1663 as in Fig. 1664, and the shear stress will be found at c (4) changing from $5\frac{1}{2}$ to $6\frac{1}{2}$ tons, and at g (13) changing from $2\frac{1}{2}$ to $3\frac{1}{2}$ tons. The pole having been taken at a distance of 20 ft., the moment scale on funicular polygon will be $\frac{3}{8}$ in. = 20 ton-feet, from which, scaling Fig. 1662, we find the bending moment at c (4) = 32 ton-feet, and at g (13) 45·5 ton-feet. Case *b*: Set off from B on Fig. 1663 three-eighths of the total load giving point y; project this point across Fig. 1664, and it will give a new datum line from which the shear stresses are to be measured. Parallel to line y on Fig. 1663 draw the line shown across the funicular polygon or bending moment diagram in Fig. 1662, which is the additional line referred to in the statement of case, and the ordinates of bending moment can be scaled, taking them as shown by shaded portions in Fig. 1665, f being the point of contrary flexure. Scaling off the moments, they will be at A = 46·7 ton-feet, and at g (13) 29·5 ton-feet.

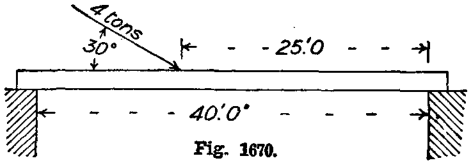


Fig. 1670.

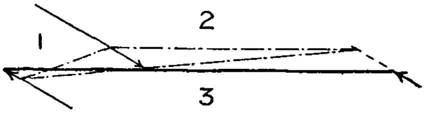


Fig. 1671.

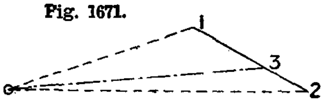


Fig. 1672.

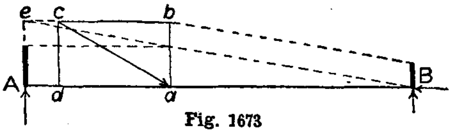


Fig. 1673

Figs. 1670 to 1673.—Diagrams showing Reaction on Abutments from Inclined Force.

Stress Diagrams for Distributed, Concentrated, and Rolling Loads.

Fig. 1666 shows the stress diagram of the bending moments and shearing forces of a plate girder, 50 ft. clear span, when submitted to a uniform dead load of $1\frac{1}{4}$ tons per ft. run. Figs. 1667 and 1668 show the same for a live or travelling load of 5 tons when at the centre and at a point 12 ft. 6 in. from the end. A very important thing to know is the effect of a live load travelling over a girder, and another diagram (Fig. 1669) is added to show this. It will be observed that the bending moments vary as for a uniformly distributed load over the whole span, but are double the amount, while the shearing stresses vary from the total live load at the abutments to one-half of the load at the centre.

Reaction on Abutments from Inclined Force.

It is sometimes required to find the reactions due to an inclined force, at the points of support of a beam resting on two abutments, as shown in Fig. 1670. Omitting the weight of the beam, which can be allowed for separately, and assuming that the friction of the beam on the abutments is sufficient to prevent it from sliding, or that it is bolted down at both ends, draw the frame diagram as shown by full lines in Fig. 1671, and number the spaces. Then draw the load line 1—2 in Fig. 1672, select a pole o, draw vectors 1—o, 2—o, and parallel to these draw lines across spaces 1 and 2 in Fig. 1671 from any point on the inclined force to meet the direction of the parallel reactions, produced if necessary. Join the extremities, and parallel to this line draw o—3 in Fig. 1672. This gives the value of the reactions 2—3 and 3—1. It may be necessary to assume that only one end of the beam is fixed as at B (Fig. 1673). Taking the inclined force as diagonal, complete the parallelogram a b c d, then the length of the horizontal side b c will be the shear on the bolt at B. Produce b c to meet a vertical line over A at point e. Join e B, and from where it cuts a b draw a horizontal line to meet e A, and from b draw a line parallel to e B, then the thickened vertical lines that are shown at A and B give the vertical reactions at the abutments.

Cast-Iron Cantilever.

The end view of a cast-iron girder used as a cantilever is shown in Fig. 1674. It is inverted to place the tension flange at the top. If the end were supported by a tie, as in Fig. 1675, the whole girder would be in compression, except for its own weight producing a slight bending moment.

Strength of Tees and Angles.

Tees or angles may be connected by a plate to form a short gangway, 5-ft. 6-in. span, being fixed either above or below, and built in at the ends or merely supported. The maximum bending moment on a beam properly fixed at the ends to make it equivalent to a continuous beam and loaded with a uniformly distributed load is $M = \frac{W L^2}{12}$, while if the beam is merely supported at the ends $M = \frac{W L^2}{8}$, the strength, or supporting power, will therefore be as 12 to 8; or a beam built in at the ends for a sufficient distance will carry half as much again as when merely supported. In Molesworth's "Pocket Book" (p. 140) a table is given for strength of tee and angle-iron when supported at the ends. This table shows that $W = \frac{K}{L^2}$, where W = safe load in lb. per ft. run, L = clear span in feet, K = coefficient as per table. The coefficients K are for 3-in. by 3-in. by ½-in. tee web downwards 7813, web upwards 6250. For 3-in. by 3-in. by ½-in. angle web downwards 5860, web upward 4688. Then, working out the four cases, the result is

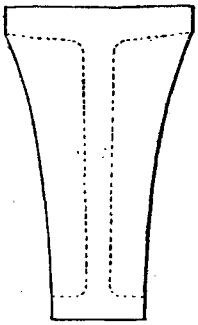


Fig. 1674.—End View of Cast-Iron Cantilever.

$$(a)\ W = \frac{7813}{5{\cdot}5 \times 5{\cdot}5} = 258{\cdot}28,$$

$$(b)\ W = \frac{6250}{5{\cdot}5 \times 5{\cdot}5} = 206{\cdot}61,$$

$$(c)\ W = \frac{5860}{5{\cdot}5 \times 5{\cdot}5} = 193{\cdot}71,$$

$$(d)\ W = \frac{4688}{5{\cdot}5 \times 5{\cdot}5} = 154{\cdot}97,$$

as the safe loads per ft. run. Multiplying these amounts by 1½ to allow for fixed ends, and by 5½ to allow for a load over the whole span, the result is (*a*) 2130, (*b*) 1704, (*c*) 1598, (*d*) 1278, as the safe total loads on one beam. Two beams will carry double these loads if the beams are not placed so far apart that the plating causes a side strain. Four men will weigh about 168 lb. × 4 = 672 lb., 672 lb. × $\frac{8}{5}$ = 1075 lb., allowing for effect of live load beyond that due to an equal dead load, so that any one of the bars singly, or even a 3-in. × 3-in. × ⅜-inch angle, would be safe with four men walking one behind the other. The most convenient arrangement would probably be that shown in Fig. 1676.

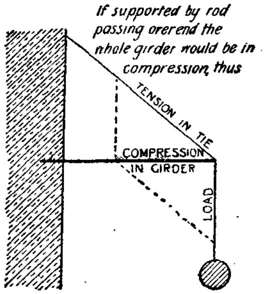

If supported by rod passing over end the whole girder would be in compression thus
TENSION IN TIE
COMPRESSION IN GIRDER
LOAD

Fig. 1675.—Cantilever with End supported by Tie.

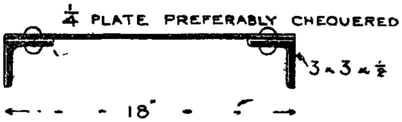


Fig. 1676.—Section showing Construction of Gangway.

Method of Finding Section and Strength of T-Iron Beam.

A T-beam of wrought iron 10 in. deep would be a very large size, and considerable difficulty would probably be experienced in obtaining such a section; but a suggestion being made that one of that depth and an equal breadth and area of 13·7 sq. in. should be used to carry a load of 4 tons distributed over a span of 20 ft., it would be worked as follows:—Allow 3 tons per sq. in. compression and 5 tons per sq. in. tension. Let b = breadth of flange, d = total depth, t = thickness, assume parallel flange and web, square angles and uniform thickness. Then

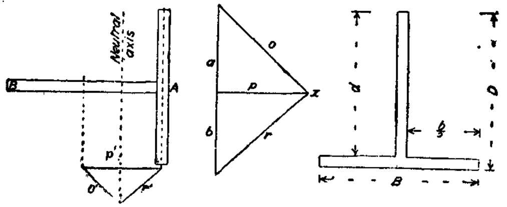


Fig. 1677. **Fig. 1678.** **Fig. 1679.**

Figs. 1677 to 1679.—Finding Strength of T-Iron Beam.

the area will be $b\,t + (d-t)\,t = b\,t + d\,t - t^2 = t\,(b + d - t)$. If the breadth is to be the same as the depth and the area is to equal 13·7 sq. in., then $t\,(10 + 10 - t) = 13{\cdot}7$, $20\,t - t^2 = 13{\cdot}7$, whence $t^2 - 20\,t = -13{\cdot}7$. Add to both sides the square of half the coefficient of t, then $t^2 - 20\,t + \left(\frac{20}{2}\right)^2 = \left(\frac{20}{2}\right)^2 - 13{\cdot}7$, and taking the square root of both sides of the equation $t - 10 = \sqrt{100 - 13{\cdot}7}$, or $t = t \pm \sqrt{86{\cdot}3} + 10$, or $t = \pm\, 9{\cdot}29 + 10$, or $t = {\cdot}71$, giving a 10 × 10 × ·71 tee iron = 13·7 sq. in. area. Checking the area by the dimensions 10 × ·71 + 9·29 × 71 = 13·6959, which is a trifle short, but ·72 thickness would be too much. To find the strength of a T-iron beam with the web downwards, thus, ⊥, first find the position of the neutral axis. This may be done graphically as shown in Figs. 1677 and 1678. Set off $a\,b$ (Fig. 1678) respectively equal to the areas A B of the beam (Fig. 1677), and join the extremities of the lines representing these with any point x called the pole. Draw the polygon $o'\,p'\,r'$ (Fig. 1677) having its sides parallel to $o\,p\,r$ respectively and intercepted between verticals drawn through the centres of gravity of A B. From the intersection of $r'\,o'$ draw a vertical line that will cut the section at the neutral axis at a distance z from the farthest edge. By calculation from Fig. 1679, $z = \frac{B\,D^2 - b\,d^2}{2\,(B\,D - b\,d)} = \frac{10 \times 10^2 - 9{\cdot}29 \times 9{\cdot}29^2}{2\,(10 \times 10 - 9{\cdot}29 \times 9{\cdot}29)} = \frac{1000 - 801{\cdot}765}{2\,(100 - 86{\cdot}3)} = \frac{198{\cdot}235}{27{\cdot}4} = 7{\cdot}235$. The next step is to find the moment of inertia I. This may be done graphically, but with rectangular sections the easier plan is to calculate the moment of inertia; thus:

$$I = \frac{(B\,D^2 - b\,d^2)^2 - 4\,B\,D\,b\,d\,(D - d)^2}{12\,(B\,D - b\,d)} =$$

$$\frac{(10 \times 10^2 - 9{\cdot}29 \times 9{\cdot}29^2)^2 - 4 \times 10 \times 10 \times 9{\cdot}29 \times 9{\cdot}29\,(10 - 9{\cdot}29)^2}{12\,(10 \times 10 - 9{\cdot}29 \times 9{\cdot}29)}$$

$$= \frac{(1000 - 801{\cdot}765)^2 - 34520 \times {\cdot}71^2}{12\,(100 - 86{\cdot}3)}$$

$$= \frac{(198{\cdot}235)^2 - 34520 \times 5041}{12 \times 13{\cdot}7}$$

$$= \frac{39297{\cdot}115 - 17401{\cdot}532}{164{\cdot}4} = 133{\cdot}185.$$

Here is now a series of equations. Effort = resistance, that is, bending moment = moment of resistance; bending moment $= \frac{W\,l}{8}$; moment of resistance (Z C) = modulus of section (Z) × modulus of rupture (C); modulus of section (Z) = moment of inertia (I) ÷ distance from neutral axis to farthest edge of section (y) $\therefore \frac{W\,l}{8} = \frac{I}{y} \times C$, whence $W = \frac{I}{y} \times C \times \frac{8}{l} = \frac{133{\cdot}185}{7{\cdot}235} \times 5 \times \frac{8}{20 \times 12} = 18{\cdot}41 \times 5 \times {\cdot}0333$ = 3·068 tons. If the T-iron were placed with the web upwards, then $W = 18{\cdot}41 \times 3 \times {\cdot}0333$ = 1·841 tons. As is often the case, the modulus of section is here taken to be the working stress in tension and compression respectively.

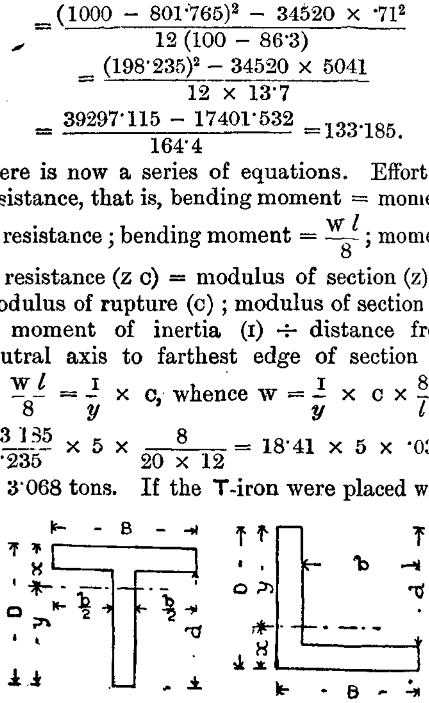


Fig. 1680. **Fig. 1681.**

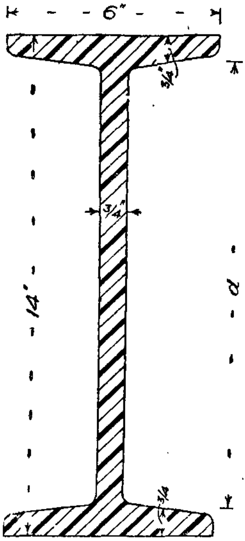
Figs. 1680 and 1681.—Finding Strength of T- and L-Section Steel.

Strength of T and L Sections.

Taking a somewhat similar case, but for steel: Let B = total breadth, D = total depth, d = net or inner depth, or depth of space, b =

net breadth of space, as shown in Fig. 1680. Then y = the distance of neutral axis from lower edge $= \frac{BD^2 - b\,d^2}{2(BD - b\,d)}$. Also I = moment of inertia of section

$$= \frac{(BD^2 - b\,d^2)^2 - 4BDb\,d(D - d)^2}{(BD - b\,d)\,12}$$



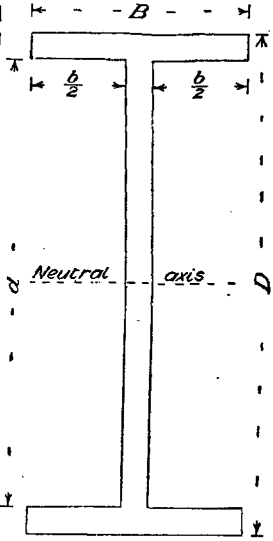


Fig. 1682.—Section of Rolled Joist. Fig. 1683.—Determining Strength of Rolled Joist by Moment of Inertia.

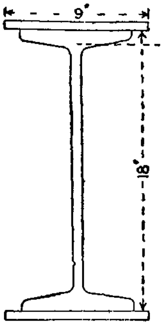
Let w = distributed loads in tons, l = span in inches, k = coefficient of reaction according to mode of loading and supporting = 8 for distributed load ends supported. Then bending moment $M = \frac{w\,l}{k}$. Let Z = modulus of section = $\frac{I}{y}$, c = modulus of rupture for transverse bending = steel tension 8 tons, steel compression $6\frac{1}{2}$ tons, then moment of resistance R in ton-inches $= \frac{I}{y} \times c$. For tee section, as Fig. 1680, $\frac{w\,l}{k} = \frac{I}{y}\,c \therefore w = \frac{I}{y} \times c \times \frac{k}{l}$. The portion y will be in tension, therefore $c = 8$. If the section had been the other way up the portion y would be in compression and c would $= 6\frac{1}{2}$, the section used as a beam showing then a falling off of $\frac{1}{5}$ of its strength. The same formulæ apply to the angle section (Fig. 1681), but in this position observe that c will be $6\frac{1}{2}$, because the web will be in compression.

Determining Strength of Rolled Joist by Moment of Inertia.

The safe distributed load for a rolled iron joist over a 20-ft. span, its depth being 14 in., and its flanges 6 in. by $\frac{3}{4}$ in., may be found by the moment of inertia as follows:—In the rolled joist shown in Fig. 1682, let M = bending moment in ton-inches under distributed load = $\frac{w\,l}{8}$, R = moment of resistance $= \frac{I}{y}\,c$, I = moment of inertia in inch units $= \frac{B\,D^3 - b\,d^3}{12}$, as in accompanying Fig. 1683, y = distance from neutral axis to farthest edge of section = $\frac{1}{2}$ depth c = limiting stress per square inch = 5 tons wrought iron, 8 tons for steel. Then for rolled iron joist 14 × 6 × $\frac{3}{4}$ metal, 20-ft. span, $M = R$, $\frac{w\,l}{8} = \frac{I}{y}c$, $w \times \frac{20 \times 12}{8} = \frac{6 \times 14^3 - 5{\cdot}25 \times 12{\cdot}5^3}{12} \times \frac{1}{7} \times 5$, $\therefore 30\,w = 517{\cdot}5 \times \frac{5}{7}$, whence $w = \frac{517{\cdot}5 \times 5}{7 \times 30} = 12{\cdot}32$ tons. But $\frac{3}{4}$ in. is unusually thick for the web; unless specially rolled it would not exceed $\frac{1}{2}$ in.

Moment of Inertia of Compound Girder.

The moment of inertia of a rolled steel joist with a plate riveted on the top and bottom will require that deductions should be made for rivet holes. The compound girder (Fig. 1684) should be divided up into rectangles as shown in Fig. 1685. Then approximately the



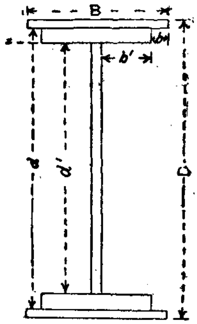
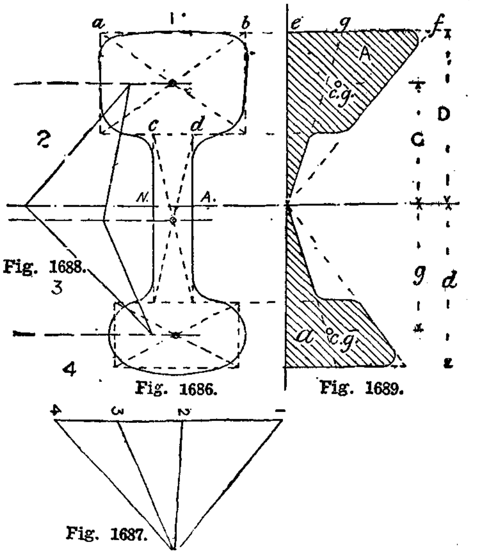


Fig. 1684.—Section of Compound Girder. Fig. 1685.—Determining Moment of Inertia of Compound Girder.

moment of inertia will be $I = \frac{B\,D^3 - 2bd^3 - 2b'd'^3}{12}$ without allowing for the rivet holes. In order to allow for the rivet holes the widths B and b′ should be diminished by the diameter of one hole, the rivets being usually placed alternately so that the section is only weakened by one hole in each flange. In this case, with ¾-in. rivets, $B = 9 - \cdot 75 = 8 \cdot 25$, $b = 1$, $b' = \frac{7 - \cdot 5625}{2} - \cdot 75 =$ say $2 \cdot 47$, $D = 19$, $d = 18$, $d' = 18 - (2 \times \cdot 9375) = 16 \cdot 125$, then $I =$



Figs. 1686 to 1689.—Determining Moment of Inertia of Bull-headed Rail.

$$\frac{8 \cdot 25 \times 19^3 - 2 \times 1 \times 18^3 - 2 \times 2 \cdot 47 \times 16 \cdot 125^3}{12} = \frac{56586 \cdot 75 - 11664 - 20712 \cdot 185}{12} = 2017 \cdot 547.$$

Moment of Inertia of 5-in. Bull-headed Rail.

The section of a bull-headed rail must first be drawn full size as shown in Fig. 1686, equalising lines for the area put round the three main portions, and horizontal lines drawn through the centre of gravity of each. Then

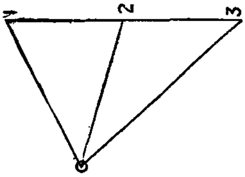


Fig. 1692.—Finding Centre of Gravity of Steel Troughing.

the reciprocal diagram (Fig. 1687) must be drawn, and the funicular polygon (Fig. 1688) to determine the position of the neutral axis N A. The inertia area diagram (Fig. 1689) may now be constructed, making the distance $e f = a b$, and $e g = c d$, completing as shown. Let A = the shaded inertia area of the upper part of the section, and a = the shaded inertia area of the lower part, D = distance from the neutral axis to the top edge of the section, d = distance from the neutral axis to the lower edge of the section, G = distance from the neutral axis to the centre of gravity of the upper inertia area, g = distance from the neutral axis to the centre of gravity of the lower

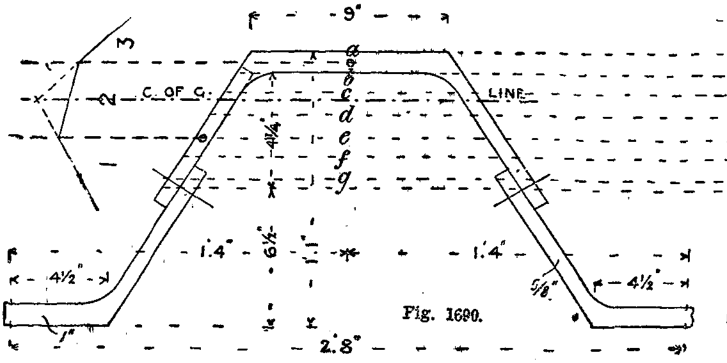


Fig. 1690.—Section of Lindsay's Steel Trough Decking.

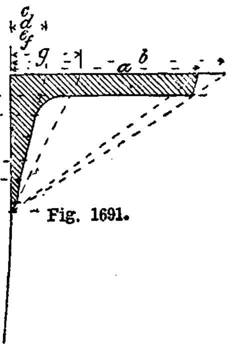


Fig. 1691.—Diagram of Inertia Area.

inertia area. Then the moment of inertia $I = DAG + dag$.

Moment of Inertia of Lindsay's Steel Troughing.

A certain heavy section of Lindsay's patent steel trough decking for bridge floors (see Fig. 1690) weighs about 67 lb. per ft. run, or 54 lb. per ft. super. riveted up. The sectional area of each trough is about 20 in By drawing the section to scale and constructing a diagram of the inertia area, as shown in Fig. 1691, the value of the inertia area may be obtained by a planimeter. In this case it appears to be 13¼ sq. in. The height of the centre of gravity may be found by graphic diagram with funicular polygon as shown in Fig. 1692. Repeating this distance on the lower side of horizontal centre line through rivets, the effective depth is obtained, which is found to be about 8½ in. Then the moment of resistance in ton-inches = the working load in tons per square inch × inertia area of section in square inches × effective depth in inches, $R = f\,a\,d = 7 \times 13\cdot25 \times 8\cdot5 = 788\cdot375$ ton-inches.

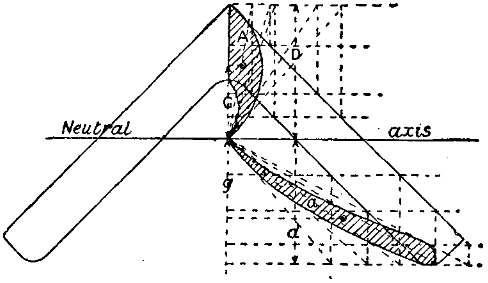


Fig. 1693.—Determining Moment of Inertia of Angle-Iron Stanchion.

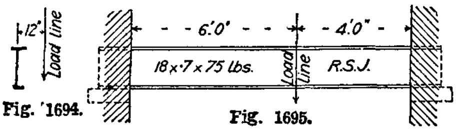


Fig. 1694. Fig. 1695.

Figs. 1694 and 1695. — Rolled Joist of which Twisting Moment is required.

Moment of Inertia of Angle-Iron used as a Stanchion.

In this case the neutral axis should be taken at right angles to the plane of easiest flexure. Let the illustration (Fig. 1693) represent the section of the angle, say 3½ in. by 3½ in. by ½ in. thick. Cut out a full-size section in drawing paper, and find the position of the centre of gravity by suspension from different points. A line through this centre of gravity, parallel with the points of the angle, will give the neutral axis. Find the inertia area, as shown by dotted lines and shaded portions, where $I = 2\,DAG + 2\,d\,a\,g = 2$ (by approximate trial only). The radius of gyration squared = moment of inertia divided by area of section. Area of 3½ in. × 3½ in. × ½ in. = 3·25 sq. in.; therefore $r^2 = \frac{2}{3\cdot25} = \cdot615$, and $r = \sqrt{\cdot615} = \cdot78$. A difference in the result will be seen if the plane of bending is assumed to lie in the direction of either of the sides of the angle, but unless there is something to compel such a failure the material would probably take the course assumed, and fail by the opening out of the jaws of the angle, reducing the radius of gyration still more. Suppose the angle-iron to be 10 ft. long and loaded exactly over the intersection of the neutral axis with the centre line, then the safe load in lb. per sq. in. will be $10000 - 33\frac{l}{r} = 10000 - 33\frac{120}{\cdot78} = 10000 - 5077 = 4923$. The area being 3·25 sq. in., the total safe axial load will be 4923 × 3·25 = 16000 lb., or, say, 7 tons; but owing to the great difficulty of ensuring the load being axial, half this amount will be ample to allow with properly fixed ends.

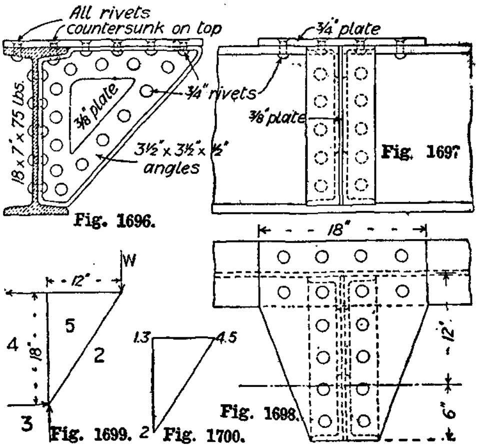


Fig. 1696. Fig. 1697. Fig. 1698. Fig. 1699. Fig. 1700.

Figs. 1696 to 1700.—Determining Twisting Moment on Rolled Joist.

Twisting Moment on Rolled Joist.

A rolled joist may occasionally have to carry an overhanging load on one side which will put a twisting moment upon it. Say, for example, a rolled steel joist, 18 in. by 7 in. by 75 lb., is loaded with 10 tons distributed, what will it carry 12 in. from centre, and what will it carry when the distributed load is omitted? The arrangement being as Figs. 1694 and 1695, the stresses will depend on how the overhanging load is applied. Assuming it to be by a bracket as in Fig. 1696 (showing section), Fig. 1697 (elevation), and Fig. 1698 (plan), the approximate working would be as follows, but no close mathematical result is possible: An 18-in. by 7-in. by 75-lb. rolled steel joist has a modulus of section of 127·6 inch-units, a distributed load of 10 tons on a span of 10 ft. produces a bending moment of $\frac{wl}{8} = \frac{10 \times 10 \times 12}{8}$ $= 150$ ton-ins., $\frac{150}{127 \cdot 6} = 1 \cdot 175$ tons per square inch, compression in top flange and tension in bottom flange. The top flange may be left out of account, as the top plate will strengthen it more than the rivet-holes weaken it, and the joist is taken as merely supported at the ends, the building-in not being sufficient to make it a fixed beam. Draw frame and stress diagrams as Figs. 1699 and 1700 for the overhanging load; then it will be seen first that the load w produces the effect of a concentrated load on the beam, which will give a bending moment of $w \times \frac{4}{10} \times 12\ (10 - 4) = 28 \cdot 8\ w$ ton-ins., making the total bending moment $150 + 28 \cdot 8\ w$ ton-ins., or a stress of $\frac{150 + 28 \cdot 8\ w}{127 \cdot 6}$ tons per square inch direct stress. There will also be the transverse thrust or load of $w \times \frac{12}{18} = \frac{2}{3}\ w$ tons, producing a horizontal bending moment of $\frac{2}{3}\ w \times \frac{4}{10} \times 12\ (10 - 4) = 19 \cdot 2\ w$ ton-ins. The modulus of section of the flange to resist this will be $\frac{b\ d^2}{6} = \frac{\cdot 928 \times 7^2}{6} = 7 \cdot 58$ inch-units for a flange of ·928 in. uniform thickness, or say 7 for actual flange, and the stress produced will be $\frac{19 \cdot 2\ w}{7}$ tons per square inch. The maximum working stress must not exceed say 7 tons per square inch; then $7 = \frac{150 + 28 \cdot 8\ w}{127 \cdot 6} + \frac{19 \cdot 2\ w}{7}$, or $7 = 1 \cdot 175 + 0 \cdot 225\ w + 2 \cdot 743\ w$, or $7 - 1 \cdot 175 = 2 \cdot 968\ w$, whence $w = \frac{5 \cdot 825}{2 \cdot 968} =$ say 2 tons overhanging weight when the beam carries 10 tons distributed. If the distributed load is omitted, the overhanging load will be found thus: $7 = \frac{28 \cdot 8\ w}{127 \cdot 6} + \frac{19 \cdot 2\ w}{7}$ or $7 = \cdot 225\ w + 2 \cdot 743\ w$, whence $w = \frac{7}{2 \cdot 968} = 2 \cdot 36$ tons.

STRENGTH OF TRUSSED BEAMS AND BRACED GIRDERS.

Combining Longitudinal and Transverse Stresses.

THE case of a beam under combined longitudinal and transverse stress is a very usual one, but not generally understood. Fig. 1701 shows a framed timber cantilever with the actual load. The bending moment produced by the transverse load is $\frac{W\,l}{8} = \frac{8\,(8 \times 12)}{8} = 96$ ton-ins., the lower 8 in the formula being a constant due to distributed load on a beam supported at the ends. The load also produces stresses in the cantilever as a whole, half the load being supported next the wall by the bolts and fastenings, and the other half at the outer point. Draw the parallelogram of forces as shown in Fig. 1702, and scale off, when it will be found that the upper member has to bear, in addition to the transverse stress, a direct pull of 7 tons. The two kinds of stress may be combined by the formula $S = \frac{W}{A} \pm \frac{M}{Z}$, where S = the stress in outer fibres in tons per square inch, W = transverse load in tons, A = sectional area in square inches, M' = bending moment in ton-ins., Z = modulus of section in inch units (often wrongly called moment of resistance in square inches). Taking 800 lb. as the extreme fibre stress for Norway pine, we have $\frac{800}{2240} = \frac{7}{b\,d} \pm \frac{96}{\frac{1}{6}\,b\,d^2}$, from which it will be found by the process of "trial and error" that a beam about 14 in. deep by 9 in. broad will be required. If it is desired to use the ordinary formula of $W = \frac{4\,b\,d^2}{L}$ and factor of safety of 5, it is necessary to revert to first principles to find its equivalent, as follows: Bending moment = moment of resistance, $\therefore \frac{W\,l}{4} = \frac{S\,b\,d^2}{6}$, $l = 12\,L$, then $W = \frac{4\ S\,b\,d^2}{6 \times 12\,L} = \frac{S\,b\,d^2}{18\,L}$; but ordinary formula is $W = \frac{4\,b\,d^2}{L}$, therefore $4 = \frac{S}{18}$, whence $S = 4 \times 18 = 72$ cwt., or 3·6 tons per square inch so called stress in extreme fibres for a breaking load. With a factor of safety of 5 this will give a working stress of 0·72 tons per square inch; and substituting this value in the equation as above, we have $0{\cdot}72 = \frac{7}{b\,d} \pm \frac{96}{\frac{1}{6}\,b\,d^2}$, from which it will appear that a beam 12 in. deep by 6 in. broad will be sufficient, but a factor of safety of 5 is only allowed in temporary work; for permanent work it should not be less than 10, which would necessitate using the 14-in. by 9-in. beam. If this case occurred in actual practice, it would be necessary to take special care in designing the joints and fastenings.

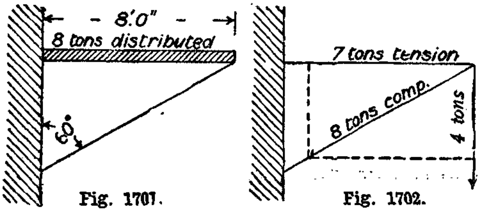


Fig. 1701. Fig. 1702.

Figs. 1701 and 1702.—Framed Timber Cantilever under Combined Longitudinal and Transverse Stress.

Stress Diagrams of Trussed Beams.

Two cases are shown in Figs. 1703 and 1704; they are both rather knotty problems. In Fig. 1703 the structure is theoretically imperfect; the upward thrust produced by the unloaded strut has no force to resist it except the internal stresses set up in the beam. To bring these under the scope of a stress diagram, show the depth of the beam over the strut,

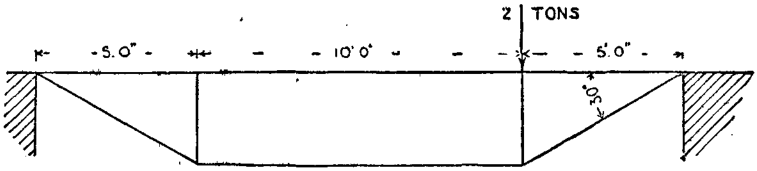

2 TONS
5.0"
10'0"
5.0"
30°

Fig. 1703.

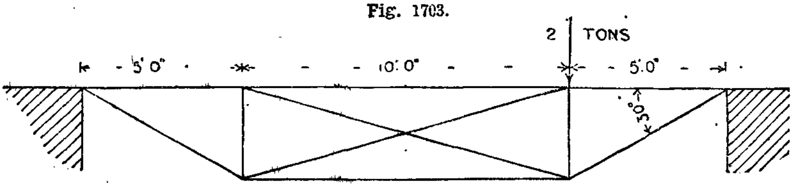

2 TONS
5'0"
10'0"
5.0"
30°

Fig. 1704.

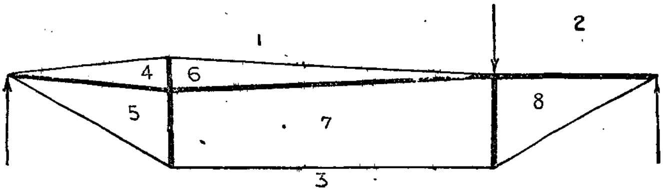


Fig. 1705.

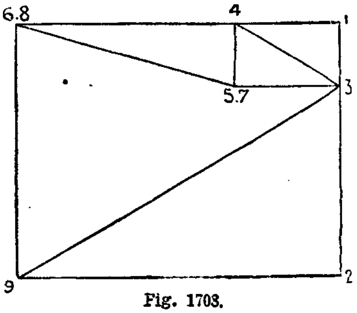


Fig. 1708.

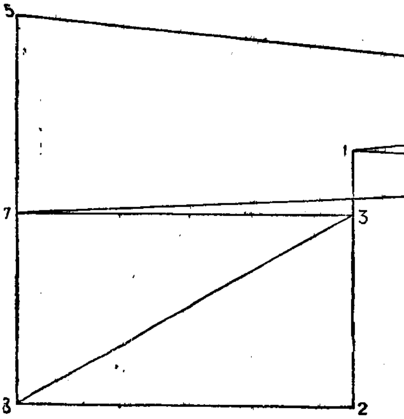


Fig. 1706.

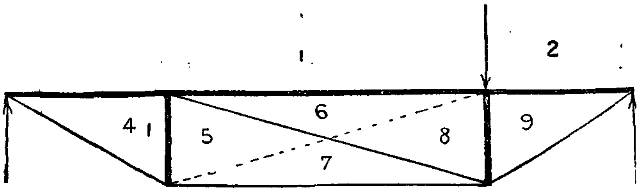


Fig. 1707.

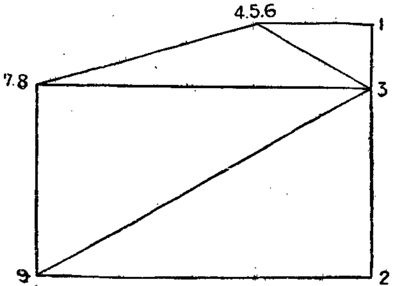


Fig. 1710.

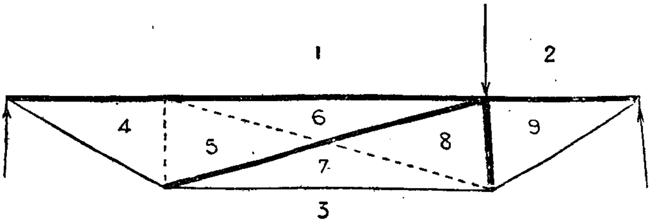
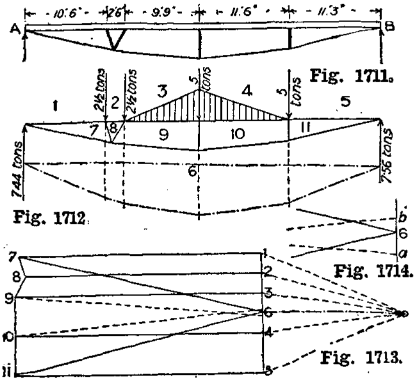


Fig. 1709.

Figs. 1703 to 1710.—Diagrams showing Stresses on Trussed Beams.

and draw the skeleton framework that might replace it, as shown in Fig. 1705, then the stress diagram will be as Fig. 1706. In Fig. 1704 a very usual case is shown, the frame diagram being the same as in Fig. 1707, and the stress diagram as Fig. 1708; but under certain structural conditions, very considerable alterations might be caused, varying anywhere between this and Figs. 1709 and 1710.



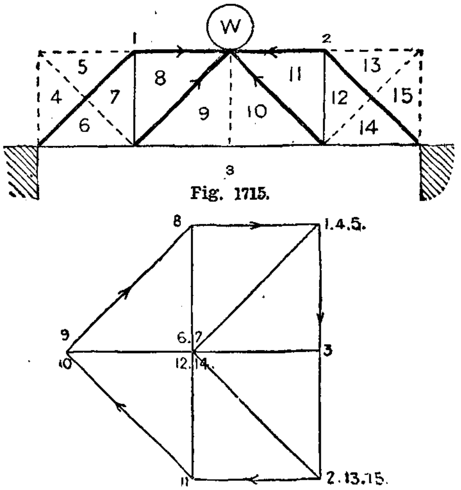
Figs. 1711 to 1714.—Stresses in Trussed Beam with Irregular Loading.

Stresses in Trussed Beam with Irregular Loading.

Judging by the preceding diagrams it will be seen that the trussed beam (shown in Fig. 1711) is a defective structure, and no stress diagram can be found directly. Whenever a trussed beam has more than one strut, the inner portions should be braced; that is, every part of the beam must be triangulated in order to get a stress diagram. On attempting to obtain the stresses in Fig. 1711, a frame diagram (Fig. 1712) is drawn, and the spaces are numbered. Then the load line in Fig. 1713 is drawn, a pole is selected, vectors are produced, and the funicular polygon is drawn on Fig. 1712 to give point 6 in Fig. 1713. Then, constructing the stress diagram Fig. 1713, it is found that 9—6 and 10—6 do not meet on the point 6, but at some distance in front of it, shown enlarged at Fig. 1714. This indicates that force 3—4 is not large enough by the amount $a\ b$ = ·75 ton, and that a beam loaded as this is would be subject to an upward bending moment on the centre span of 3·96 ton-feet over the middle strut, reducing towards the intermediate struts, as shown by vertical shade lines. With dead loads and no bracing, it is necessary that the struts should have a certain length under each load, otherwise this distorting action will occur. To avoid this bending moment, the centre load may be increased to 5·75 tons, and a practically complete stress diagram can then be drawn. This, however, will not be strictly accurate, except where the loads are symmetrical both in amount and position, although not necessarily all equal. With a rolling load such a beam must be braced

Determining Stresses in Girders.

The nature of the stress in any member of a braced girder is found in the same way as in a roof truss. The example in Fig. 1715 is not a very practical form, but is here shown to scale with its accompanying stress diagram (Fig. 1716) for the purpose of showing how the nature of the stress in any member is discovered. Take the members meeting under the load: 1—2 in the frame diagram acts downwards, therefore put an arrow-head in that direction on 1—2 in the stress diagram. Now, going round in the direction of the hands of a watch, take 2—11 and follow on in the stress diagram with concurrent arrow-heads. Transfer the direction of the arrow-head to 2—11 in the frame diagram; then, as it points towards the joint, the member is in compression. Next take 11—10 in the stress diagram, and



Figs. 1715 and 1716.—Determining Stresses in Braced Girder.

transfer the direction to the frame diagram, when it also will be found acting towards the joint and indicating compression. Points 10 and 9 are identical in the stress diagram, therefore there is no stress in member 10—9. Continue with 9—8 and 8—1 ; both these will be found in compression, and that completes the members meeting at the point of application of the load. To ascertain the same particulars with regard to the other members, select another joint—say, where 6, 7, 8, 9, 3 meet. Rub out the previous arrow-heads, and start with a member in which the nature of the stress is known, say, 8—9. Then, in the stress diagram, put an arrow-head on 8—9, acting towards the new joint—that is, the opposite way on to its previous position—and follow with arrow-heads on 9—3, 3—6, 6—7, 7—8, transferring these to the frame diagram, and discovering that 9—3 will be tension because the force acts away from the joint, 3—6 tension, 6—7 no stress, 7—8 tension. Any other joint may be tested in the same way, always working from the known to the unknown.

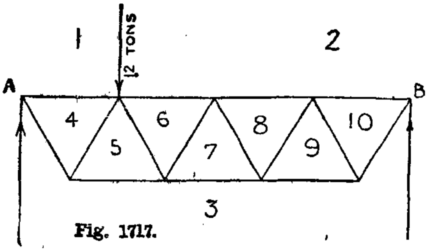


Fig. 1717.

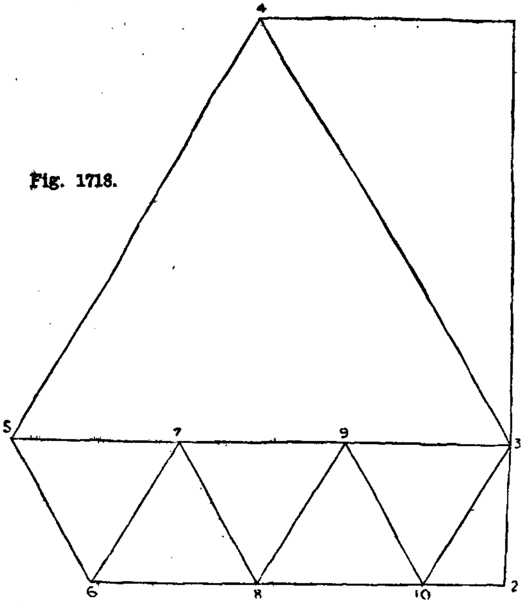


Fig. 1718.

Figs. 1717 and 1718.—Frame and Stress Diagrams for Warren Girder.

Stresses in Warren Girder.

The frame diagram being as Fig. 1717, the only difficulty in commencing the stress diagram for this case (Fig. 1718) is to find the reaction at each abutment. It may be done graphically, but it is almost self-evident from the position of the load—namely, three-fourths of 12 tons at A, and one-fourth of 12 tons at B, being inversely proportional to the distances from the load to abutment. Then work in the order of the numbered points. In a frame diagram (Fig. 1719) for a Warren girder of five bays, the bars in compression are marked with double lines, those in tension with single lines, and those without stress are dotted. The girder is badly designed for a central load on the bottom flange, as a cross strain will occur upon the middle portion of the bottom flange in addition to the tensile stress. The reciprocal diagram (Fig. 1720) shows the direct stresses in the various members produced by this load.

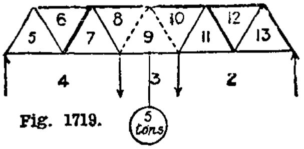


Fig. 1719.

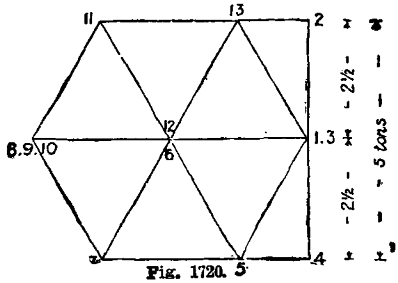


Fig. 1720.

Figs. 1719 and 1720.—Frame and Reciprocal Diagrams of Five-bay Warren Girder.

Stresses in Warren Girder Irregularly Loaded.

The following is the simplest way to work this case graphically, although not the usual

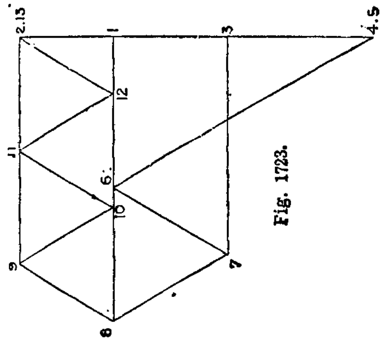

2.13
1
3
4.5
12
11
6
10
9
7
8

Fig. 1723.

Figs. 1721 to 1723.—Stresses in Warren Girder Irregularly Loaded.

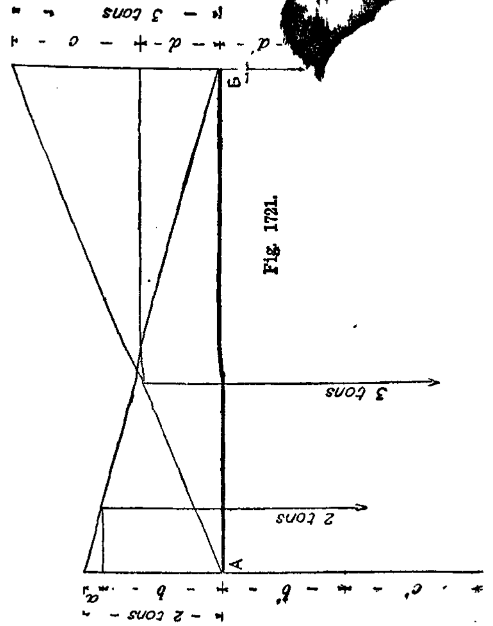

3 tons
2 tons
A
B
a
b
c
d
3 tons
2 tons

Fig. 1721.

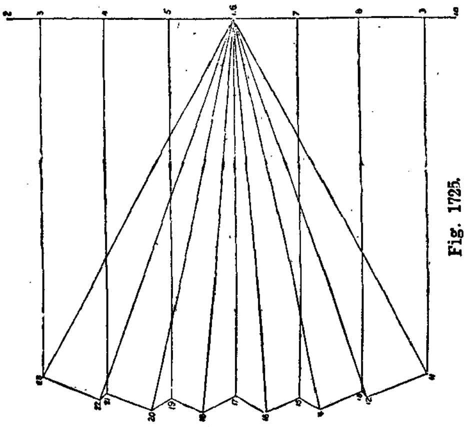


Fig. 1725.

Figs. 1724 and 1725.—Stresses in Bowstring Girder.

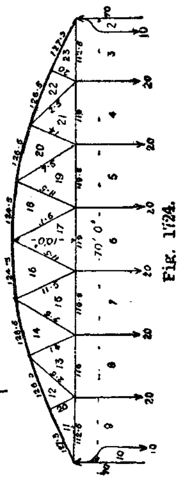

70' 0"
10
20
20
20
20
20
20
20
10

Fig. 1724.

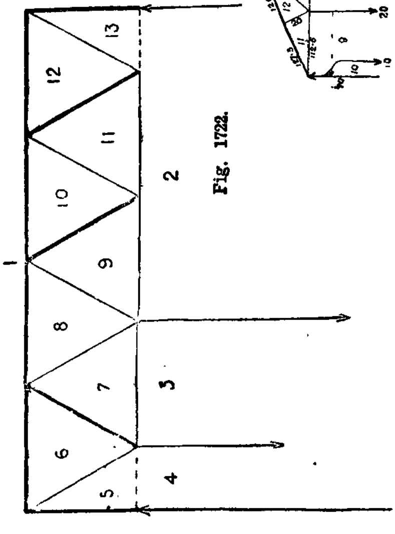

1
2
3
4
5
6
7
8
9
10
11
12
13

Fig. 1722.

method. Let A B (Fig. 1721) be the span of the girder to scale, with the loads shown at the correct distances from A. At A set up to scale a vertical equal to the weight of 2 tons, and join the extremity of the vertical with a point B. Draw a vertical from the weight to meet the last line, and then draw a horizontal to cut the vertical from A, giving *a* the pressure on support B, and *b* the pressure on support A produced by the load. In a similar manner set up a vertical line to represent 3 tons over point B, join the extremity with A, draw a vertical from the weight of 3 tons to meet the line, and then draw a horizontal to meet the vertical from B, dividing the vertical so that *c* represents the pressure produced at A by the load, and *d* represents the pressure on B. These may now be transferred below A and B as a^1, b^1, c^1, d^1, and represents the reactions of supports. Now the frame diagram (Fig. 1722) may be drawn, and the reciprocal diagram (Fig. 1723). From the latter the stresses may be scaled off in the usual way.

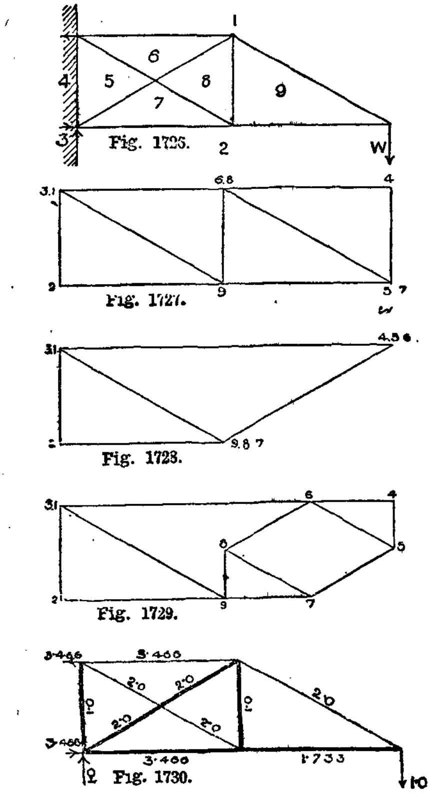


Fig. 1725.

Fig. 1727.

Fig. 1728.

Fig. 1729.

Fig. 1730.

Figs. 1726 to 1730.—Stresses in Trussed Cantilever.

Stresses in Bowstring Girder.

Taking Fig. 1724 as the frame diagram of an ordinary bowstring girder, 70-ft. span and 10ft. deep, carrying a distributed load of 2 tons per foot run on the lower flange, Fig. 1725 will be the stress diagram, and the stress values scaled off will be as marked on Fig. 1724. If the curve were a true parabola, the stress diagram correctly drawn, and the values accurately scaled, the stress in the flanges would generally be considered to be nearly uniform throughout, although, as figured on the illustration, there are substantial differences.

Stresses in Trussed Cantilever.

This case is similar in principle to that of a lattice girder with vertical members. In that girder the usual assumption is that the vertical bars transmit half the load hanging from them or resting [illegible] them; but this assumption may be more [illegible] erroneous, according to the workman[illegible] the girder. In the present case sti[illegible]om for doubt perhaps exists, althou[illegible] is apparently so simple. The accom[illegible]strations show the different possib[illegible]. 1726 shows the frame diagram, ta[illegible] for the thickened members and unity [illegible] Fig. 1727 shows the stress diagram on t[illegible]supposition that bar 6—8, 5—7 is not acting; Fig. 1728 shows the stress diagram, when bars 5—6, 7—8, and 8—9 are out of use. One of these being absent, the other, as will be seen, is necessarily inoperative. Fig. 1729 shows the stress diagram when half the load is passing down bar 8—9 and the other half is transmitted through 6—8, 5—7. Fig. 1730 shows the maximum stresses produced on the members by the various conditions detailed above. Suppose the frame to be put together with turned pins in drilled holes, an exact fit, the fact that there must be some definite strain in each of the members is quite evident, but opinion varies as to the result. Some American writers have devoted many pages of mathematics to the elucidation. The ordinary reciprocal diagram is powerless to combine the effects when there is a redundant member in the truss, and Willett's system of substituted members in the frame diagram does not appear to be of any service.

Strength of Rolled Joist Bracket.

It often happens that an addition has to be made to a building and carried on brackets, as for instance a lavatory with four water-closets that is to be erected at the first-floor level of an existing building. The structure is to be carried on cantilevers with brackets or struts. The floor will be of concrete 8 in. thick, and the walls of corrugated iron on light steel framing. Fig. 1731 and Fig. 1732 show the plan and section of the proposed work. The deeper that the strut is carried and the nearer to the point of the bracket, the less will be the stress in the different parts. Fig. 1733 shows the frame diagram with the stresses marked on the different members for the arrangement with the strut at 30 degrees to the floor level and reaching to the point. The load may be taken as ¼ ton per foot run distributed on each rolled joist; the stress diagram will then be as shown in Fig. 1734, and a bending moment of $\frac{w\,l^2}{8} = \frac{\frac{1}{4} \times 7 \times 7}{8} = 1{\cdot}53$ ton-ft. will be produced by the distributed load, as added to Fig. 1734. This is for the outer brackets. The middle bracket will have the same load on the rolled joist and double the load on the strut, because the intermediate joists are to have no strut, but to be connected by a joist across their ends. The horizontal part has direct stress = 1·5 tons tension and bending moment = 1·53 ton-ft. = 18·36 ton-ins. that have to be provided for. The maximum stress is given by the formula $\frac{W}{A} \pm \frac{M}{Z}$. Assume 6-in. by 3-in. by 13-lb. rolled steel joist with a modulus of section (sometimes improperly called moment of resistance in square inches) of 6·92 and the sectional area of 3·8 sq. in., of which about 1·3 will be in one flange. Then $\frac{W}{A} \pm \frac{M}{Z} = \frac{1{\cdot}5}{1{\cdot}3} + \frac{18{\cdot}36}{6{\cdot}92} = 3{\cdot}8$ tons per square inch maximum stress, which is well on the safe side. The outer struts have 1·75 tons compression, and the unsupported length is, say, 8 ft. Try a 3-in. by 2½-in. by ⅜-in. tee. Then by Gordon's formula

$$\frac{fs}{1 \times \frac{1}{a}\left(\frac{l}{d}\right)^2} = \frac{26 \times 1{\cdot}9^2}{1 + \frac{1}{750} \times \left(\frac{96}{2\frac{1}{2}}\right)^2} = \text{say 16 tons}$$

crushing weight, and with a factor of safety of 6 gives 2·7 tons safe load, so that this section will be sufficient. For the central strut with double load a 3½-in. by 3½-in. by ⅜-in. tee will be suitable. Another arrangement is shown in the frame diagram Fig. 1733, where the strut is at 45 degrees to the floor level and starts at

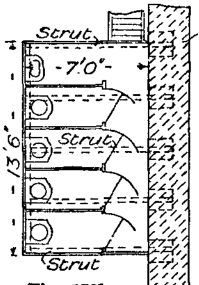

Strut
-7'0"-
Strut
-13'6"-
Strut
Wall 2'9 thick

Fig. 1731.

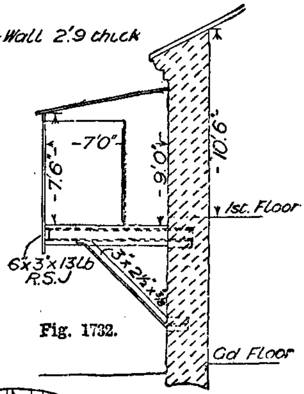

-7'0"-
-7'6"-
-9'0"-
-10'6"-
1st. Floor
6x3x13Lb R.S.J
3"x2½"x⅜
Gd Floor

Fig. 1732.

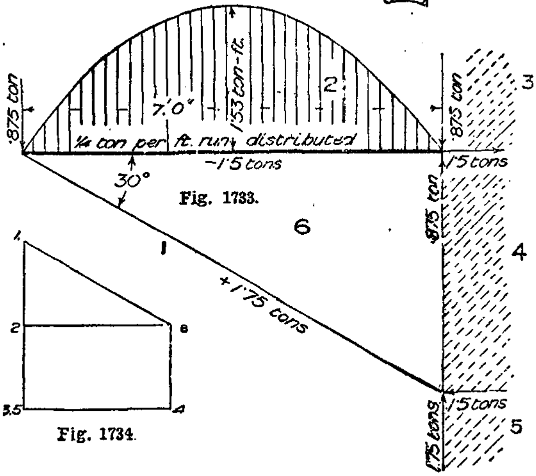

·875 ton
1·53 ton-ft.
7'0"
2
3
¼ ton per ft. run distributed
-1·5 tons
30°
·875 ton
1·5 tons
·875 ton
6
1
4
+1·75 tons
1·5 tons
1·75 tons
5
1
2
6
3.5
4

Fig. 1733.

Fig. 1734.

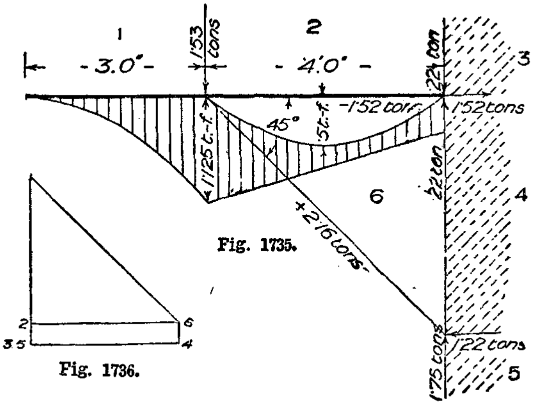

1
1·53 tons
2
2·2 ton
3
-3'0"-
-4'0"-
-1·52 tons
1·52 tons
45°
·5 t-f.
1·125 t-f.
2·2 ton
6
4
+2·16 tons
1·22 tons
1·75 tons
5
2
6
3.5
4

Fig. 1735.

Fig. 1736.

Figs. 1731 to 1736.—Diagrams showing Strength of Rolled Joist Bracket.

4 ft. from the fixed end of the cantilever. The bending moment under the strut due to the load on the overhanging portion will be $\frac{\text{W L}}{2} = \frac{3 \times 3}{4 \times 2} = 1{\cdot}125$ ton-ft. The bending moment at the fixed end will be $\frac{3 \text{ W L}}{32} = \frac{3 \times 1 \times 4}{32} = {\cdot}375$ ton-ft., and the maximum bending moment due to the load between the point of support and the fixed end will be $\frac{wl^2}{8} = \frac{\frac{1}{4} \times 4 \times 4}{8} = {\cdot}5$ ton-ft., the complete bending moment diagram being as shown by the shaded portion in Fig. 1735. The stress diagram is shown in Fig. 1736, and the stresses are marked on the members in Fig. 1735, + denoting compression and − tension. In this case the maximum bending moment only need be taken account of, and the formula $\text{M} = \frac{\text{I}}{y}$ c is used. Assume 6-in. by 3-in. by 13-lb. rolled steel joist with a moment of inertia of 20·77. Then $1{\cdot}125 \times 12 = \frac{20{\cdot}77}{3}$ c, whence c = say 2, so that this section will probably do. For the strut try a 3-in. by $2\frac{1}{2}$-in. by $\frac{3}{8}$-in. tee. Then by Gordon's formula $\frac{f s}{1 + \frac{1}{a}\left(\frac{l}{d}\right)^2} = \frac{26 \times 1{\cdot}92}{1 + \frac{1}{750} \times \left(\frac{70}{2\frac{1}{2}}\right)^2} =$ say 24 tons crushing weight, and with a factor of safety of 6 gives 4 tons safe load, so that this section will be suitable.

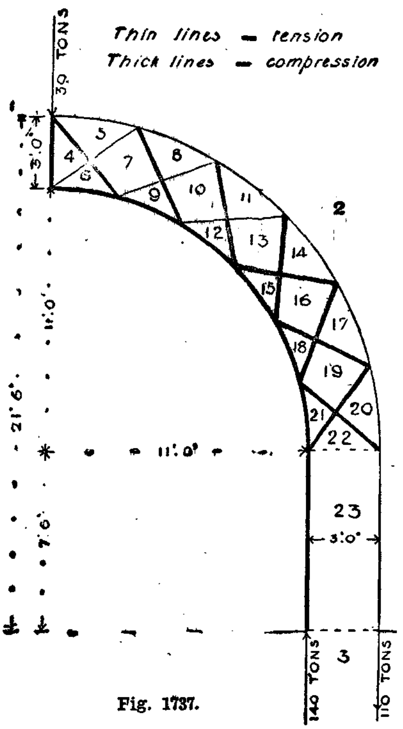


Fig. 1737.

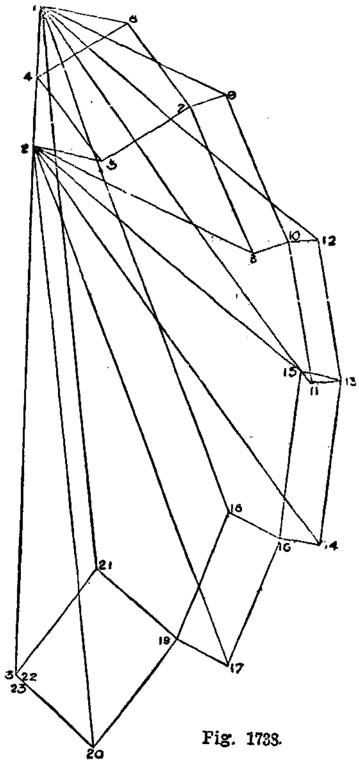


Fig. 1738.

Figs. 1737 and 1738.—Diagrams showing Strength of Bent Lattice Cantilever.

Strength of Bent Lattice Cantilever.

Assuming the outline to be as shown in Fig. 1737, the stresses will be as shown in Fig. 1738. To start the stress diagram, half the load (of 30 tons) may be taken as passing down the vertical bar 1 − 4 = 15 tons, then by leverage the stress 2 − 3 will be $\frac{30 \times 11}{3} = 110$ tons,

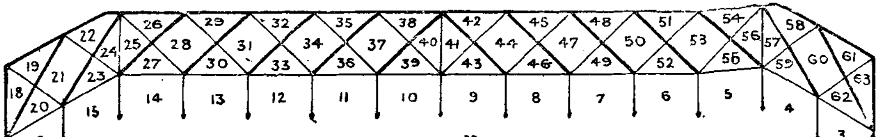

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Fig. 1739.

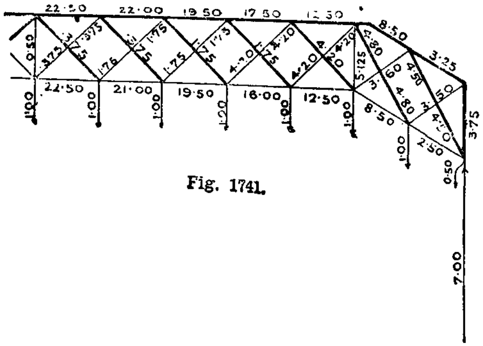
22·50
22·00
19·50
17·50
12·50
8·50
3·25
3·75
7·00
0·50
22·50
21·00
19·50
16·00
12·50
8·50
2·50
1·00
1·75
4·20
5·125
4·80
4·50

Fig. 1741.

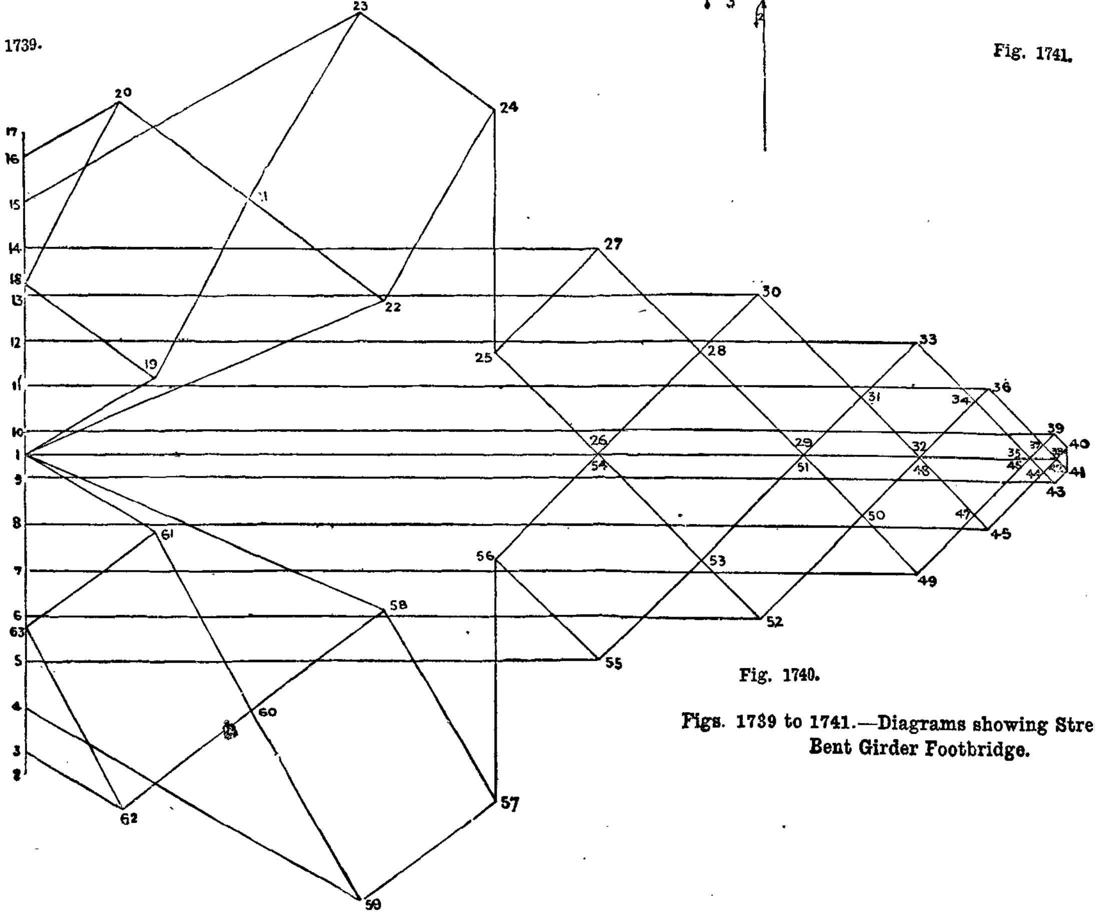
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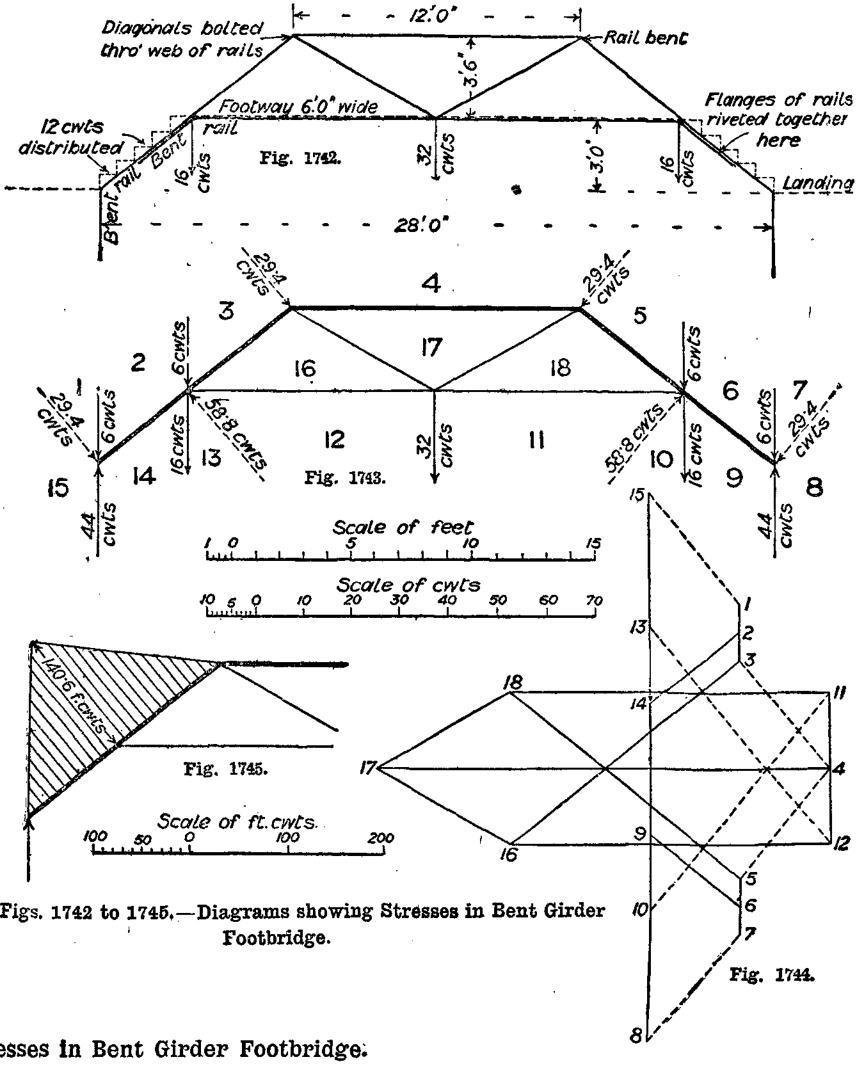
Fig. 1740.

Figs. 1739 to 1741.—Diagrams showing Stresses in Bent Girder Footbridge.

and that in 3 − 1 will be 30 + 110 = 140 tons. The remainder of the work is simple. The stresses will be greatly reduced if the cantilever be widened at the ground line and tapered upwards.

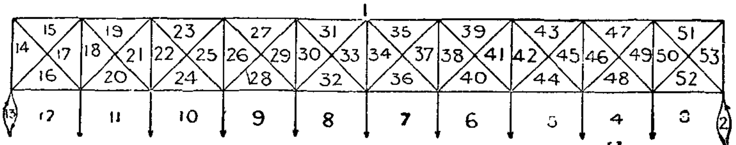

Diagonals bolted thro' web of rails
12'0"
Rail bent
3'6"
Footway 6'0" wide
rail
12 cwts distributed
Bent rail
Bent
16 cwts
Fig. 1742.
32 cwts
Flanges of rails riveted together here
3'0"
16 cwts
Landing
28'0"
29.4 cwts
4
29.4 cwts
3
5
6 cwts
17
2
16
18
6 cwts
1
6
7
29.4 cwts
6 cwts
58.8 cwts
12
11
6 cwts
29.4 cwts
16 cwts
13
32 cwts
58.8 cwts
10
16 cwts
15
14
Fig. 1743.
9
8
44 cwts
44 cwts
Scale of feet
1 0 5 10 15
Scale of cwts
10 5 0 10 20 30 40 50 60 70
15
1
13
2
3
18
14
11
140.6 ft.cwts
Fig. 1745.
17
4
Scale of ft.cwts.
100 50 0 100 200
9
12
16
5
10
6
7
Fig. 1744.
8

Figs. 1742 to 1745.—Diagrams showing Stresses in Bent Girder Footbridge.

Stresses in Bent Girder Footbridge.

The bent girder footbridge shown in Fig. 1739 carries a distributed load. If desired, additional bars may be inserted to make a trellis girder instead of a simple lattice girder. The stresses for unity load on each bay, scaled from the stress diagram (Fig. 1740), have been marked upon a half elevation of this girder (Fig. 1741), compression being shown by thick lines and tension by thin lines.

Another Case.—In the case of a footbridge to be built (except the two diagonals) of railway metals, the stresses may be determined graphically on the same principle as a collar-beam roof truss with yielding abutments. Fig. 1742 shows the general arrangement, and Fig. 1743 the frame diagram. The dotted lines are the virtual forces produced by the leverage of the lower ends, which cause a bending moment on both sides, as shown by the ordinates in the diagram for one side given in Fig. 1745, and taken up by the stiffness of the outer inclined members. These virtual forces must be found before a stress diagram can be made; thus 44 cwt., reaction − 6 cwt. directly over = 38 cwt. balance acting upwards. Then 38 cwt. vertical × 3·75 ft. leverage distance perpendicular to direction ÷ 4·875 ft. length of actual lever = 29·2 cwt. acting at right angles to lever. Taking the whole lever as rigid, this will cause a reaction of similar amount at the far end, and double the amount in the centre, as shown in Fig. 1743. Then number all the spaces, internal and external, including the dotted virtual forces, and draw the stress diagram as in Fig. 1744. In determining the stresses on a braced column it will not require calculating as a whole if the length be under six diameters. The bracing on all four sides reduces the calculation to the piece of channel bar in one bay of the height. This must be taken as having the full load, and the least radius of gyration should be taken in the formula $p = \dfrac{8000}{1 + \dfrac{l}{30000\ r^2}}$, where p = safe load per square inch of section, l = unsupported length in inches, r = least radius of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2

Fig. 1746.

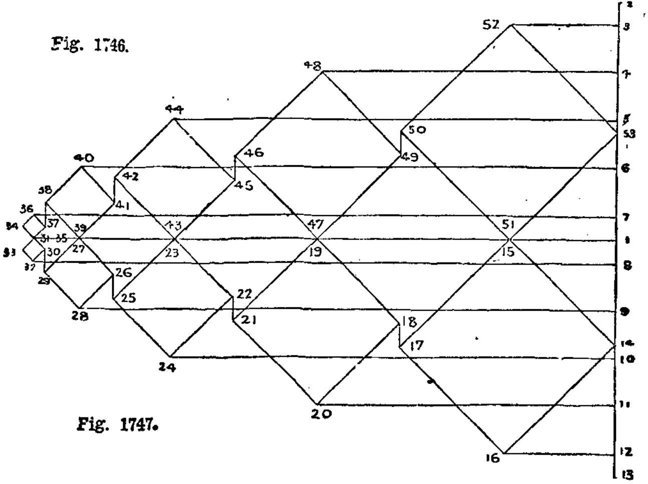
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Fig. 1747.

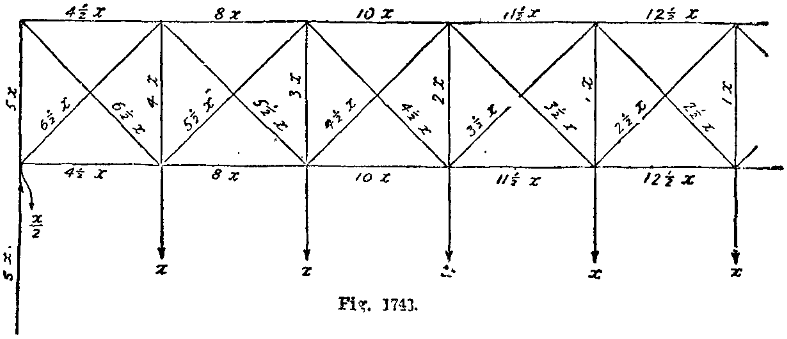
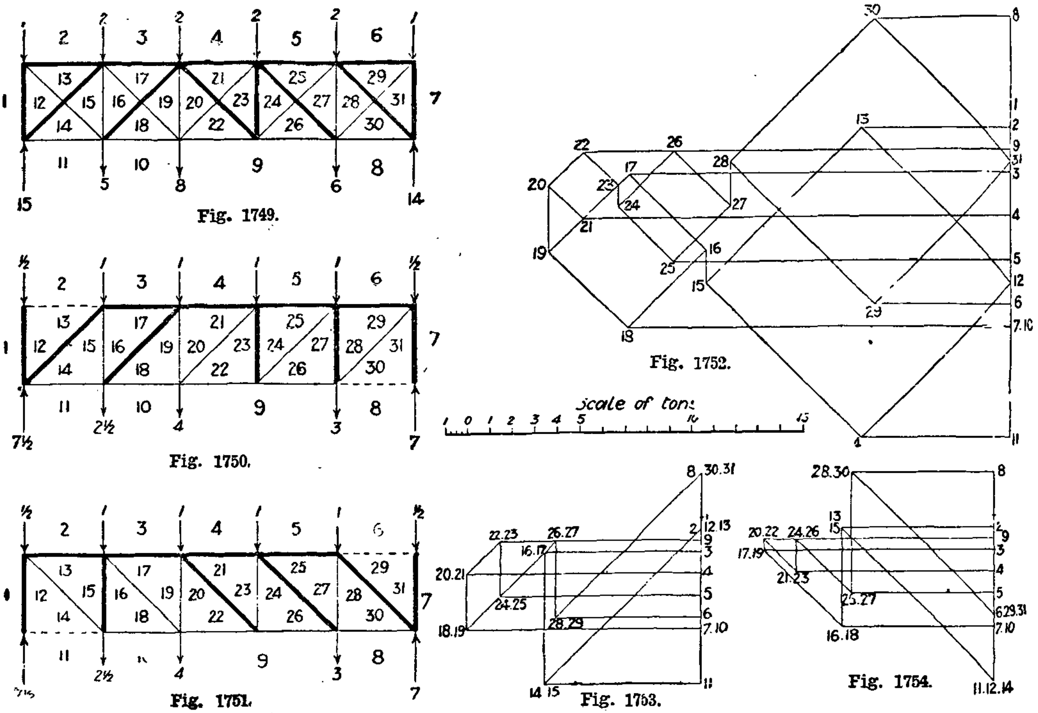

4½ x
8 x
10 x
11¼ x
12½ x
5 x
x/2
x
x
x
x
x

Fig. 1748.

Figs. 1746 to 1748.—Diagrams showing Stresses in Lattice Girder with Verticals.

gyration in inches. The section may first be found approximately by allowing 4·75 tons per square inch for 10 diameters long, 4 tons for 20 diameters, 3·25 tons for 30 diameters, and 2·5 for 40 diameters. The horizontal and diagonal members are not amenable to calculation at present, and must be designed from the sense of fitness.

the various parts assist in carrying the load. With unit load (x) per bay the maximum possible stresses under a uniformly distributed dead load, allowing for unavoidable imperfections in workmanship, will be approximately as shown in Fig. 1748, being in excess of the results given by any single stress diagram, and cannot be formed into a reciprocal diagram.



Figs. 1749 to 1754.—Diagrams showing Stresses in Lattice Girder with Vertical Bars and Irregular Loading.

Stresses in Lattice Girder with Verticals.

Fig. 1746 shows the frame diagram of lattice girder with load on bottom flange. The common assumption, for which there is hardly sufficient justification in fact, makes half the load on each apex pass through the vertical bar. Fig. 1747 shows the stress diagram on this basis. If the load were upon the top flange there would still be half the load assumed to pass through the verticals, and the stresses would be identical in amount throughout the girder, but of opposite character in the verticals. The actual stresses depend very much upon the workmanship, as this type of girder contains redundant parts, and it is practically impossible to say to what extent

Lattice Girder with Vertical Bars and Irregular Loading.

The stresses in this girder (Fig. 1749) will be easier to determine if it be separated into the two component girders (Figs. 1750 and 1751). Construct the reciprocal diagrams (Figs. 1753 and 1754), and then take the algebraical sum of the stresses for the construction of Fig. 1752. It will then be seen that the stress in 1—12 consists of 1—2 + $\frac{4}{5}$ of $\frac{2-3}{2}$ + $\frac{3}{5}$ of $\frac{3-4}{2}$ + $\frac{2}{5}$ of $\frac{4-5}{2}$ + $\frac{1}{5}$ of $\frac{5-6}{2}$ + $\frac{1}{5}$ of $\frac{8-9}{2}$ + $\frac{3}{5}$ of $\frac{9-10}{2}$ + $\frac{4}{5}$ of $\frac{10-11}{2}$.

Stresses in Trellis Girder.

In computing the stresses in the various members of a trellis girder it is not necessary to make a separate diagram for each system of trussing or triangulation; a single reciprocal diagram will cover the whole case, the only difficulty being to find the stress in the end verticals. In the frame diagram (Fig. 1755) it will be seen that the pull from 12—14 is due to half the load 5 — 6, and besides this half load there is load 1—2 to add, which enables 1—12 to be marked off on the line of loads in the stress diagram. The part 1—13 has to transmit the thrust from 12—15, the pull from 13—15, and the load from above; the thrust from 12—15 is derived from seven-eighths of load 2—3, and three-eighths of load 6—7; the pull from 13—15 is derived from five-eighths of 4—5 and one-eighth of 8—9; then there is the load from above, which, added to the loads transmitted by thrust and pull, enable 1—13 to be marked off. The remainder of the stress diagram presents no difficulty, and will be as shown in Fig. 1756.

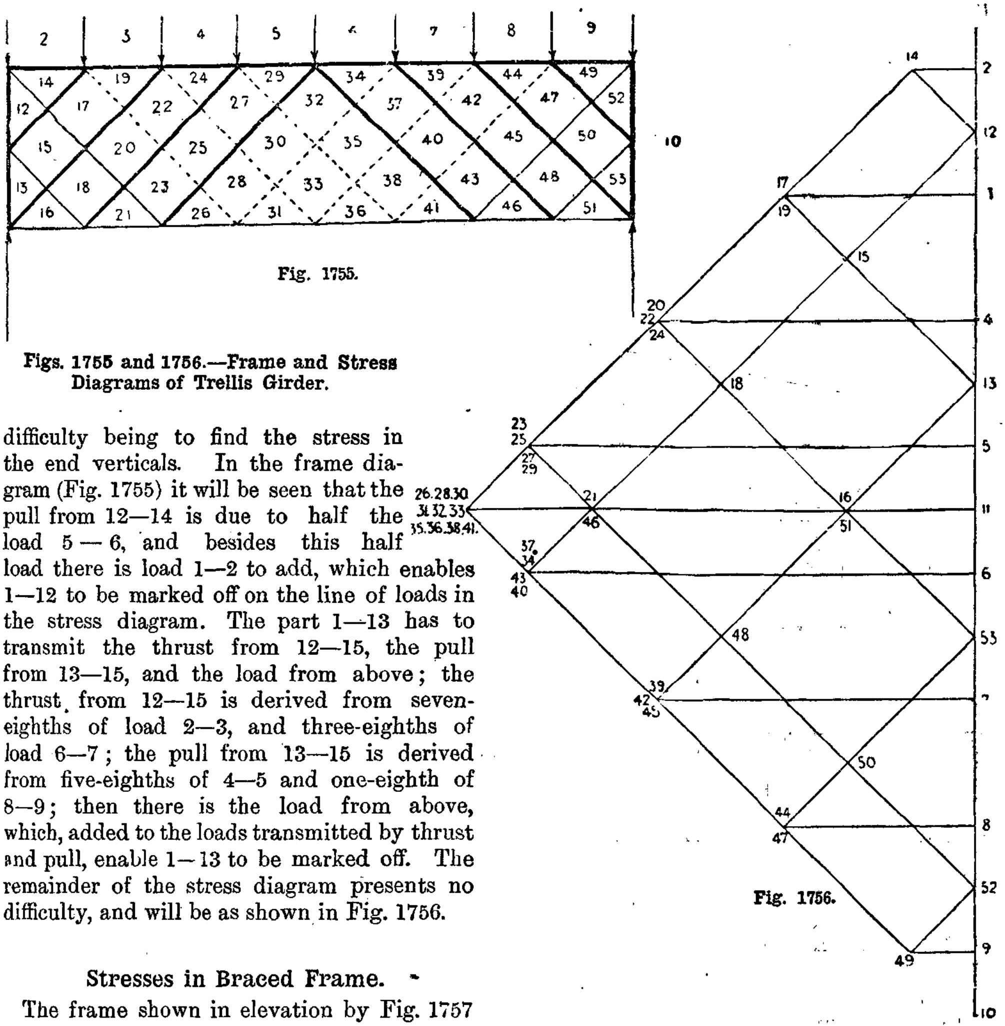


Fig. 1755.

Fig. 1756.

Figs. 1755 and 1756.—Frame and Stress Diagrams of Trellis Girder.

Stresses in Braced Frame.

The frame shown in elevation by Fig. 1757 has its own weight to carry, besides an additional load above, and the wind on one side. Fig. 1757 shows the direction lines of the forces and reactions. Assume this frame to be divided into two component structures, Figs. 1758 and 1759, each taking half the load, and from these structures construct the stress diagrams. A funicular polygon must be used in each case, in order to find the position of point 8 on the reciprocal diagram. This polygon should be constructed in the manner shown in Fig. 1757, but, in order to economise space, has not been repeated on Figs. 1758 and 1759. The frame diagrams can be marked for tension and compression by thin and thick lines, by observing the sequence of the forces in the stress diagrams, and then the algebraic sum of the stresses from Figs. 1761 and 1762 can be used for constructing Fig. 1760. With sufficient practice, Fig. 1760 might be constructed directly from Fig. 1757, but the safer plan is to adopt the method shown. This, however, is strictly an indeterminate structure, and the actual distribution of stress is dependent on the uniformity or otherwise of the workmanship at all the joints.

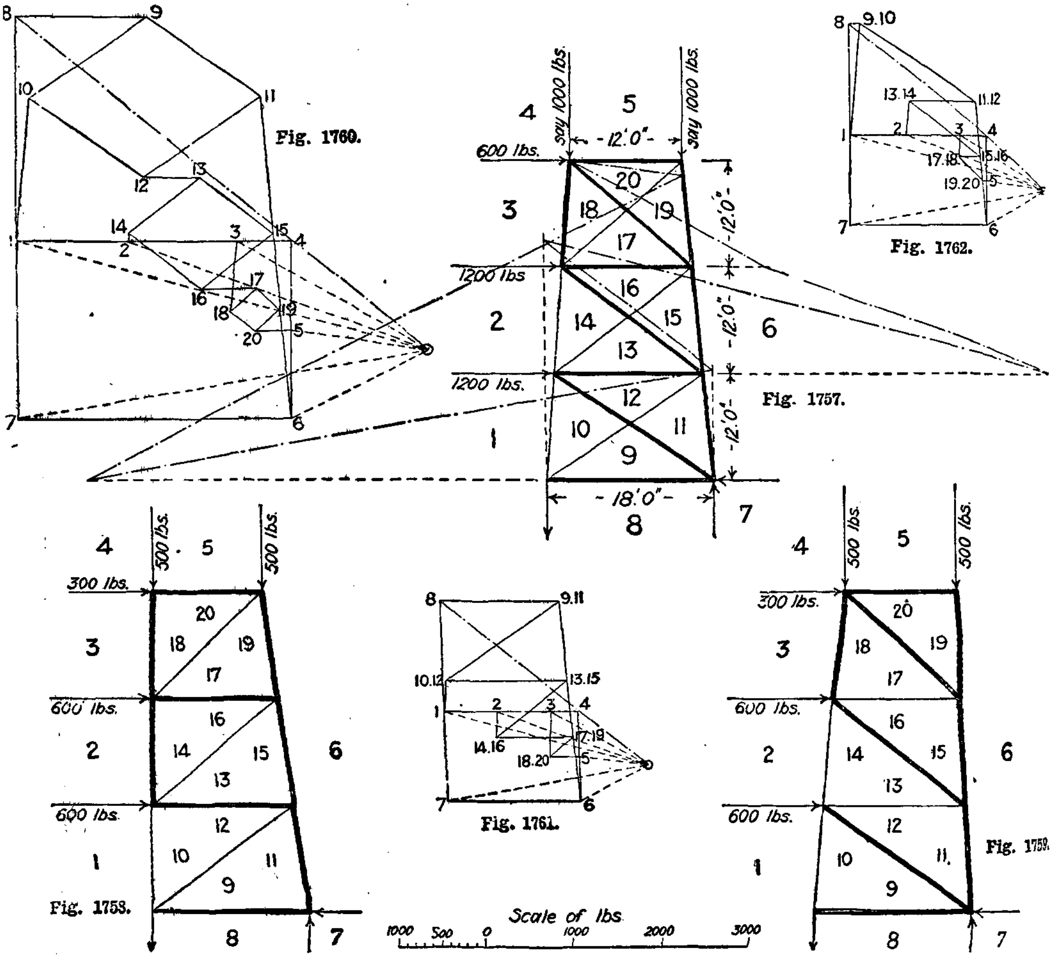


Fig. 1760.
Fig. 1757.
say 1000 lbs.
say 1000 lbs.
600 lbs.
1200 lbs.
1200 lbs.
-12'0"-
-12'0"-
-12'0"-
-12'0"-
-18'0"-
Fig. 1762.
Fig. 1758.
500 lbs.
300 lbs.
600 lbs.
600 lbs.
Fig. 1761.
Scale of lbs
1000
500
0
1000
2000
3000
Fig. 1759.

Figs. 1757 to 1762.—Diagrams showing Stresses in Braced Frame.

Reactions in Supports of Continuous Beams.

It is assumed that a continuous beam rests with its ends supported, and is also supported by a central column, and the question arises, How are the reactions on these supports investigated? The reactions of the supports for continuous beams are determined by means of the "theorem of three moments," which is a very troublesome piece of calculation. A much better idea can be obtained of the conditions under which a continuous beam exists by taking a small lattice girder and investigating it by means of reciprocal diagrams. This is a matter of such interest and importance to engineers and others that a full series for the simplest case is given in Figs. 1763 to 1780. It will be seen that the girder is assumed to have a uniformly distributed load of 48 tons spread along the top flange, which gives the forces shown above the girder. The actual distribution of stress in a continuous girder, and the reactions of the supports, depend upon the relative levels of the latter, so that a settlement of foundation is a very serious matter. By assuming that the girder is in the first

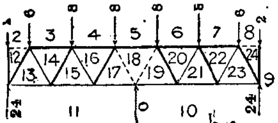

4 6 8 8 8 8 6 2
2 3 4 5 6 7 8
12 14 16 18 20 22 24
13 15 17 19 21 23
1 9
24 11 0 10 24

Fig. 1763.

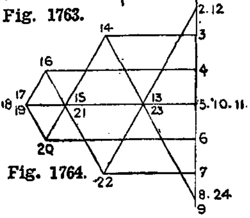

1
2.12
3
4
5.10.11
6
7
8.24
9
14 16 17 18 19 15 21 13 23 20 22

Fig. 1764.

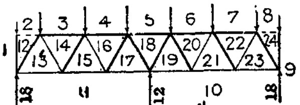

2 3 4 5 6 7 8
12 14 16 18 20 22 24
13 15 17 19 21 23
1 9
18 11 12 10 18

Fig. 1767.

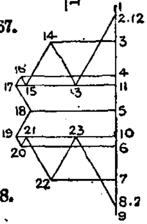

1
2.12
3
4
11
5
10
6
7
8.2
9
14 16 17 15 13 18 19 21 23 20 22

Fig. 1768.

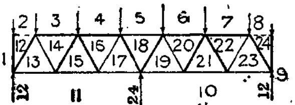
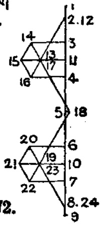

2 3 4 5 6 7 8
12 14 16 18 20 22 24
13 15 17 19 21 23
1 9
12 11 24 10 12

Fig. 1771.


1
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18
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7
8.24
9
14 15 13 17 16 20 21 19 23 22

Fig. 1772.

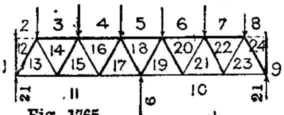
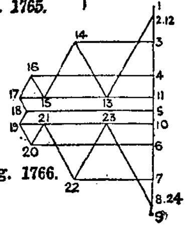

2 3 4 5 6 7 8
12 14 16 18 20 22 24
13 15 17 19 21 23
1 9
21 11 6 10 21

Fig. 1765.


1
2.12
3
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7
8.24
9
14 16 17 15 13 18 21 23 19 20 22

Fig. 1766.

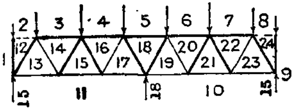
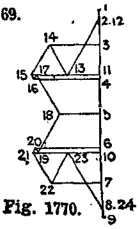

2 3 4 5 6 7 8
12 14 16 18 20 22 24
13 15 17 19 21 23
1 9
15 11 18 10 15

Fig. 1769.


1
2.12
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7
8.24
9
14 17 13 15 16 18 20 21 19 23 22

Fig. 1770.

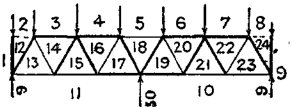

2 3 4 5 6 7 8
12 14 16 18 20 22 24
13 15 17 19 21 23
1 9
9 11 30 10 9

Fig. 1773.

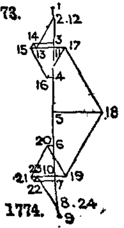

1
2.12
14 3 17
15 13 11
16 4
5 18
20 6
23 10 19
21 22 7
8.24
9

Fig. 1774.

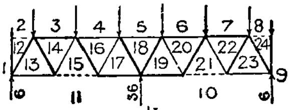

2 3 4 5 6 7 8
12 14 16 18 20 22 24
13 15 17 19 21 23
1 9
6 11 36 10 6

Fig. 1775.

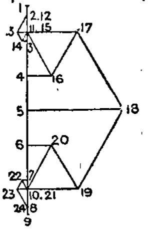

1
2.12
11.15 17
13
14 3
4 16
5 18
6 20
22 7
23 10.21 19
24 8
9

Fig. 1776.

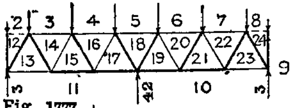
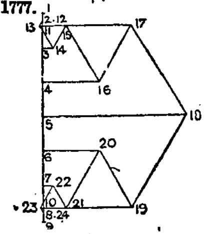

2 3 4 5 6 7 8
12 14 16 18 20 22 24
13 15 17 19 21 23
1 9
3 11 42 10 3

Fig. 1777.


1
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13 11 15 17
3 14
4 16
5 10
6 20
7 22
23 10 21 19
8.24
9

Fig. 1778.

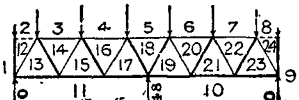

2 3 4 5 6 7 8
12 14 16 18 20 22 24
13 15 17 19 21 23
1 9
0 11 48 10 0

Fig. 1779.

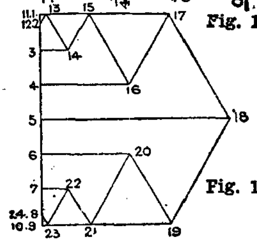

11.1.
122
13 15 17
3 14
4 16
5 18
6 20
7 22
24.8
10.9
23 21 19

Fig. 1780.

Figs. 1763 to 1780.—Diagrams showing Reactions in Supports of Continuous Beams.

instance carried by the end supports only, and then that the central column is gradually raised to cause it to take more and more load until the girder is entirely supported by it, all possible conditions are indicated, and the corresponding reciprocal diagrams not only show the distribution of stress throughout the girder under these changing conditions, but show that at a certain ratio between the reactions the stresses in the girder are on the whole at a minimum. With a girder continuous over two spans as in this case, the minimum stress occurs when the outer supports are each carrying $\frac{3}{16}$ of the load and the central support $\frac{5}{8}$, as in Figs. 1773 and 1774, and this is the distribution given in Rankine, Molesworth, Rivington, etc., for a beam continuous over two spans. If it is wished to apply this method to a beam of three equal spans, a lattice girder of 9 bays with 60 tons distributed, might be taken and a series of diagrams constructed with the reactions from 15 tons each to reactions of 5 tons on the end supports and 25 tons on the others.

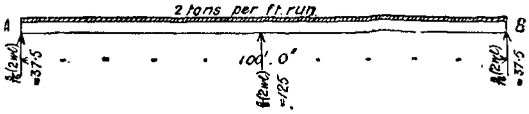


Fig. 1781.

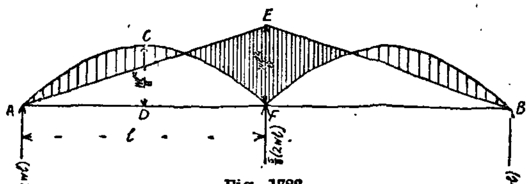


Fig. 1782.

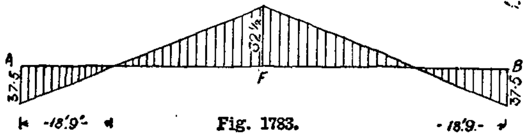


Fig. 1783.

Figs. 1781 to 1783.—Diagrams showing Stresses on Continuous Girder.

Stresses on Continuous Girder.

According to Humber's "Handy Book of Strains," the bending moment will vary in the two cases of a beam of uniform and equal section and a beam of uniform strength, but the manner in which the section can affect the question of bending moment, which in other cases is decided by the span, the mode of fixing or supporting, and the position and amount of the loads, is not very perceptible. It is probably a question of relative deflection. Continuous girders, and most other girders of large span, are in practice made of uniform strength, and Humber's formulæ for bending moment in a girder supported at both ends and in the centre, and carrying a uniformly distributed load, as shown in Fig. 1781, give the diagram as shown in Fig. 1782, where A C F = parabola,

Fig. 1784.

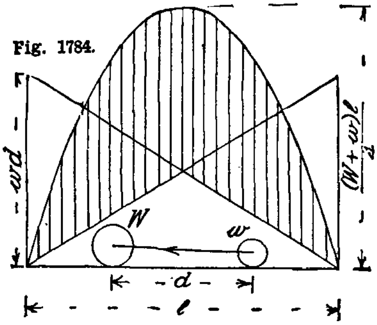


Fig. 1785.

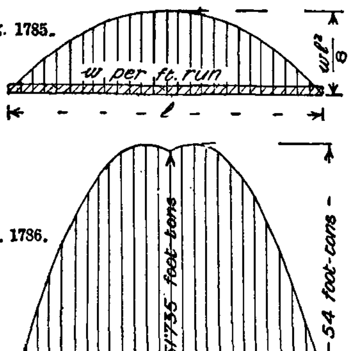
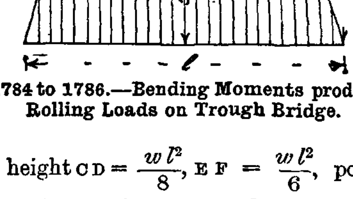


Fig. 1786.



Figs. 1784 to 1786.—Bending Moments produced by Rolling Loads on Trough Bridge.

whose height C D = $\frac{w\,l^2}{8}$, E F = $\frac{w\,l^2}{6}$, point of contrary flexure from A = $\frac{3}{4}l$. The load on supports will, by the theorem of three moments, be A = $\frac{3}{16}$ of total, F = $\frac{5}{8}$, B = $\frac{3}{16}$, whence $\frac{3}{16} \times 200 = 37\cdot5$ for A, $\frac{5}{8} \times 200 = 125$ for F, and $\frac{3}{16} \times 200 = 37\cdot5$ for B. The shearing force diagram will be as shown in Fig. 1783, the distance of point of no stress from A being $\frac{37\cdot5}{125} \times 50 = 15$ ft.

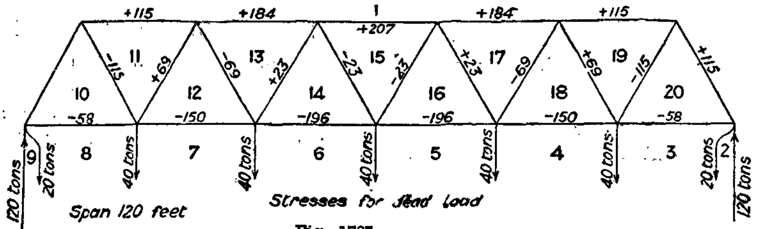

+115
+184
1
+207
+184
+115
11
13
15
17
19
-115
+69
-69
+23
-23
-23
+23
-69
+69
-115
+115
10
12
14
16
18
20
-58
-150
-196
-196
-150
-58
9
8
7
6
5
4
3
2
120 tons
20 tons
40 tons
40 tons
40 tons
40 tons
40 tons
20 tons
120 tons
Span 120 feet
Stresses for dead load

Fig. 1787.

Dead load of 2 tons per ft. run

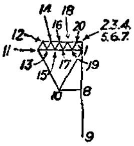

14 18
12 16 20 2.3.4.
11 5.6.7.
13 17 1
15 19
10 8
9

Fig. 1788.

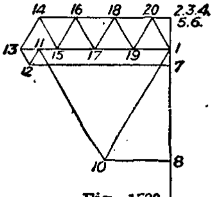

14 16 18 20 2.3.4.
5.6.
13 11 15 17 19 1
12 7
10 8
9

Fig. 1789.

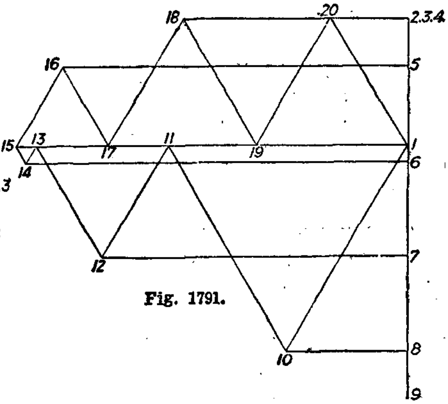

18 20 2.3.4
16 5
15 13 17 11 19 1
14 6
12 7
10 8
9

Fig. 1791.

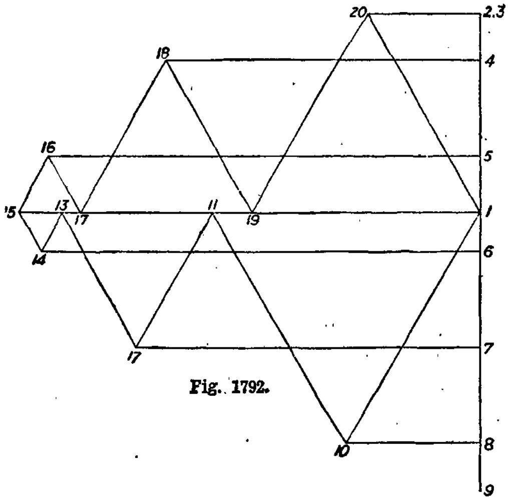

20 2.3
18 4
16 5
15 13 17 11 19 1
14 6
17 7
10 8
9

Fig. 1792.

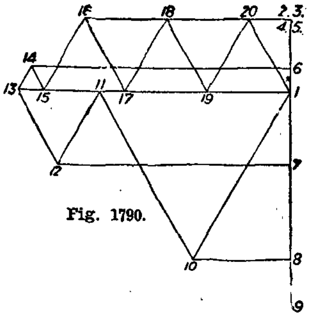

16 18 20 2.3.
4 5.
14 6
13 15 11 17 19 1
12 7
10 8
9

Fig. 1790.

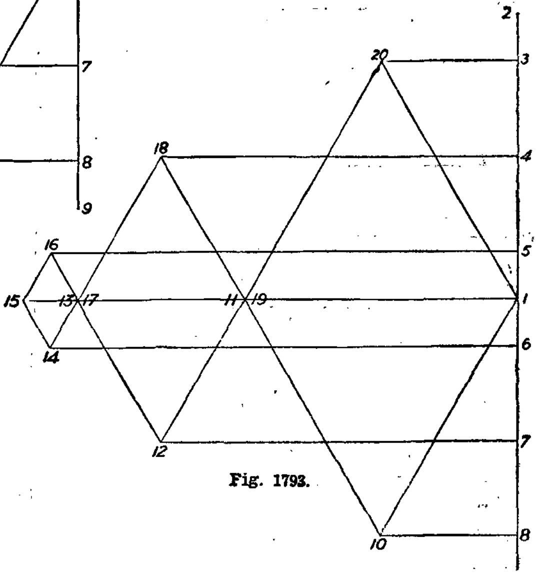

2
20 3
18 4
16 5
15 13 17 11 19 1
14 6
12 7
10 8
9

Fig. 1793.

Figs. 1787 to 1793.—Stresses produced by Rolling Load on Warren Lattice Girder.

Rolling Loads on Trough Bridge.

The maximum bending moments that are produced by rolling loads of 5 tons and $3\frac{1}{2}$ tons at a fixed distance of 11 ft. 8 in. apart on a span of 26 ft. will be as shown in Fig. 1784, where $\frac{(W + w)l}{4} = \frac{(5 + 3{\cdot}5)\,26}{4} = 55{\cdot}25$ ft.-tons, and $w\,d = 3{\cdot}5 = 11{\cdot}66 = 40{\cdot}83$ ft.-tons. To this must be added the bending moment diagram due to the dead load that is produced by the weight of the troughing and of the roadway. An average of 1 cwt. per ft. super. allowed for the weight of the road material and the filling, and $\frac{1}{2}$ cwt. per ft. super. allowed for the troughing, would give for a width of 2 ft. 8 in. of double trough $(1 + \frac{1}{2}) \times 2{\cdot}66 \div 20 = 0{\cdot}2$ ton per foot run, as shown in Fig. 1785, where $\frac{wl^2}{8} = \frac{0{\cdot}2 \times 26^2}{8} = 16{\cdot}9$ ton-ft. Adding these together, the diagram Fig. 1786 is produced, the bending moment M in the centre being $55{\cdot}25 - \frac{40{\cdot}83}{2} + 16{\cdot}9 = 51{\cdot}735$ ton-ft., and the maximum at 2 ft. each side of the centre by scale = 54 ton-ft., or 648 ton-in. The best section of trough girder that would meet this case appears to be Dorman, Long and Co.'s D Medium, with a moment of resistance of 673 ton-in.

Rolling Load on Lattice Girder.

Rolling loads produce a much greater complication of stresses than dead loads; but where a rolling load is to be provided for, there is no help for it but to make the necessary investigation. The case may be solved graphically or by calculation. The former method will probably be the more convincing, but it is desirable to study both. Fig. 1787 shows a Warren lattice girder of 120-ft. span, with a rolling load of 2 tons per foot run on the bottom flange, occupying a length at least equal to the span. Assuming the load to enter on the girder from the left-hand side, Figs. 1788 to 1798 show reciprocal stress diagrams when the load reaches each apex until the whole length of girder is covered, and then when the tail-end of the load leaves each apex until the girder is clear. These stresses, being scaled off, may be tabulated as shown in Fig. 1799, + signifying compression and − tension. The maximum value of each stress on each bar must now be collected

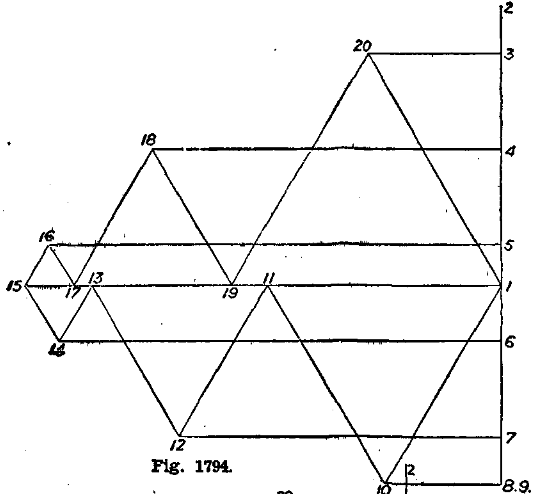


Fig. 1794.

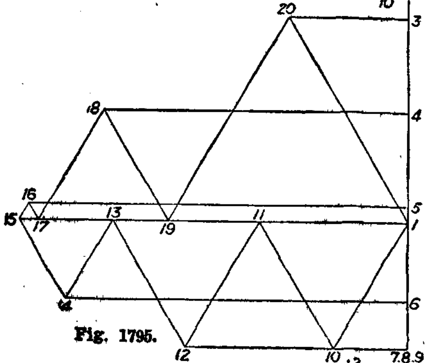


Fig. 1795.

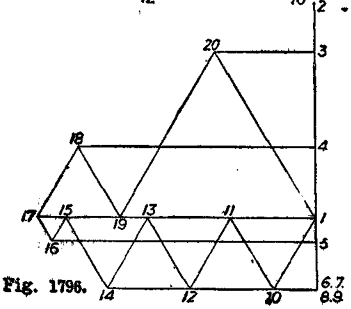


Fig. 1796.

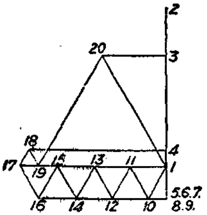


Fig. 1797.

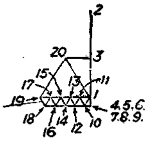


Fig. 1798.

Figs. 1794 to 1798.—Stresses produced by Rolling Load on Warren Lattice Girder.

in two columns on the right, as shown, and the girder may be designed from these. That is to say, taking bar 13—14, it must be capable of bearing either 15 tons tension or 38 tons compression. The labour by this method is, of course, very great, but the designer feels tolerably safe, because he can see the principle of it all through. By calculation, first, for the horizontal bars or flanges, we have the formula $s = \frac{x}{l}(l - x) \operatorname{cosec} \theta \, \frac{1}{2} w \, n$, where s = maximum stress of bar in tons, x = distance of centre of bar from left-hand abutment in feet, l = span in feet, θ = angle between web bracing and flange in degrees, w = load in tons per foot run, n = number of bays in girder. Then, for the inclined bars or web bracing, we have the formula $s = m \operatorname{cosec} \theta$, where s = stress in tons, and m = the ordinate in tons passing from base C B through the apex of the pair of bars under consideration to the semi-parabola of shear stress A E or C D produced by the rolling load, shown in Fig. 1800. These ordinates may be obtained thus, $m = \frac{w(l-x)(l-x)}{2l}$, so that the complete formula will be $s = \pm \frac{w(l-x)(l-x)}{2l} \operatorname{cosec} \theta$, namely + for bars inclined downwards to left abutment, from which the start must be made and carried

Member	Load over bays numbered as below											By scaling		By calculation	
	8	8 and 7	8 to 6	8 to 5	8 to 4	8 to 3	7 to 3	6 to 3	5 to 3	4 to 3	3	Max. ten.	Max. com.	Max. ten.	Max. com.
1-10	+20	+54	+81	+100	+112	+115	+95	+61	+35	+15	+4	-	115	0·96	116·43
10-11	-20	-54	-81	-100	-112	-115	-95	-61	-35	-15	-4	115	-	110·43	0·96
10-8	-10	-27	-41	-50	-56	-58	-47	-31	-18	-8	-2	58	-	63·51	-
11-1	+20	+54	+81	+100	+112	+115	+95	+61	+35	+15	+4	-	115	-	115·47
11-12	-4	+8	+35	+54	+65	+69	+74	+61	+35	+15	+4	4	74	8·66	77·94
12-7	-18	-58	-98	-127	-144	-150	-131	-94	-53	-23	-6	150	-	155·88	-
12-13	+4	-8	-35	-54	-65	-69	-74	-61	-35	-15	-4	74	4	77·94	8·66
13-1	+16	+62	+116	+154	+177	+184	+168	+124	+70	+31	+8	-	184	-	184·75
13-14	-4	-15	-11	+8	+19	+23	+28	+38	+35	+15	+4	15	38	24·05	47·15
14-6	-14	-54	-110	-158	-186	-196	-182	-143	-87	-39	-10	196	-	202·07	-
14-15	+4	+15	+11	-8	-19	-23	-23	-38	-35	-15	-4	38	15	47·15	24·05
15-1	+12	+46	+105	+163	+196	+207	+196	+163	+105	+46	+12		207	-	207·85
15-16	-4	-15	-35	-38	-28	-23	-19	-8	+11	+15	+4	38	15	47·15	24·05
16-5	-10	-39	-87	-143	-182	-196	-186	-158	-110	-54	-14	196	-	202·07	-
16-17	+4	+15	+35	+38	+28	+23	+19	+8	-11	-15	-4	15	38	24·05	47·15
17-1	+8	+31	+70	+124	+168	+184	+177	+154	+116	+62	+15	-	184	-	184·75
17-18	-4	-15	-35	-61	-74	-69	-65	-54	-35	-8	+4	74	4	77·94	8·66
18-4	-6	-23	-53	-94	-131	-150	-144	-127	-98	-58	-18	150	-	155·88	-
18-19	+4	+15	+35	+61	+74	+69	+65	+54	+35	+8	-4	4	74	8·66	77·94
19-1	+4	+15	+35	+61	+95	+115	+112	+100	+81	+54	+20	-	115	-	115·47
19-20	-4	-15	-35	-61	-95	-115	-112	-100	-81	-54	-20	115	-	116·43	0·96
20-3	-2	-8	-18	-31	-47	-58	-56	-50	-41	-27	-10	58	-	63·51	-
20-1	+4	+15	+35	+61	+95	+115	+112	+100	+81	+54	+20	-	115	0·96	116·43

Fig. 1799.

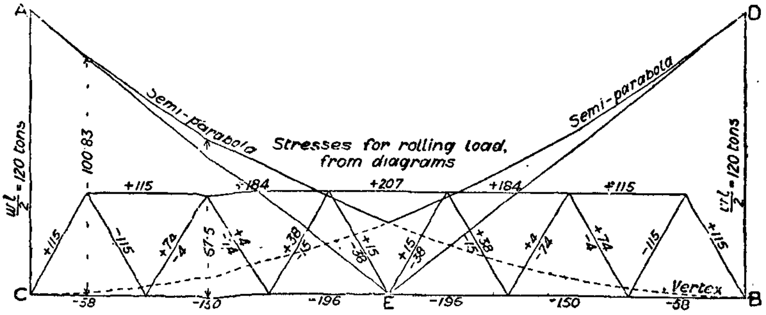


Fig. 1800.

Figs. 1799 and 1800.—Stresses produced by Rolling Load on Warren Lattice Girder.

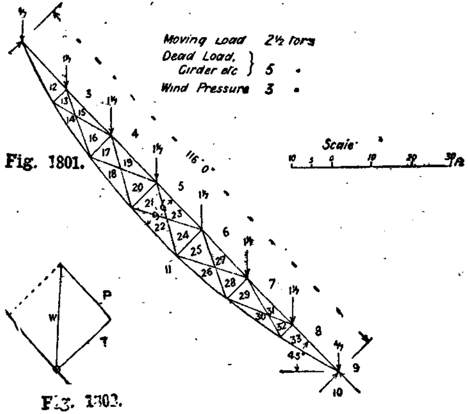

Moving Load 2½ tons
Dead Load, Girder etc } 5 "
Wind Pressure 3 "
Scale
10 5 0 10 20 30 ft
116' 0"
4½
1¼
3
4
5
6
7
8
9
10
11
45°
W
P
T

Fig. 1801.

Fig. 1802.

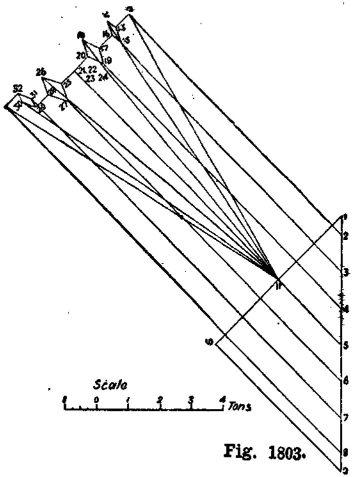

Scale
1 0 1 2 3 4 Tons
1
2
3
4
5
6
7
8
9
10
11

Fig. 1803.

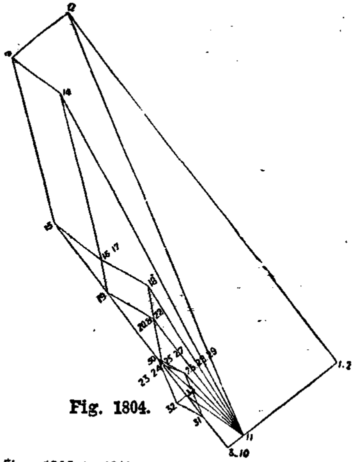
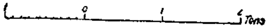


Fig. 1804.

Scale for Figs. 1805 to 1810.



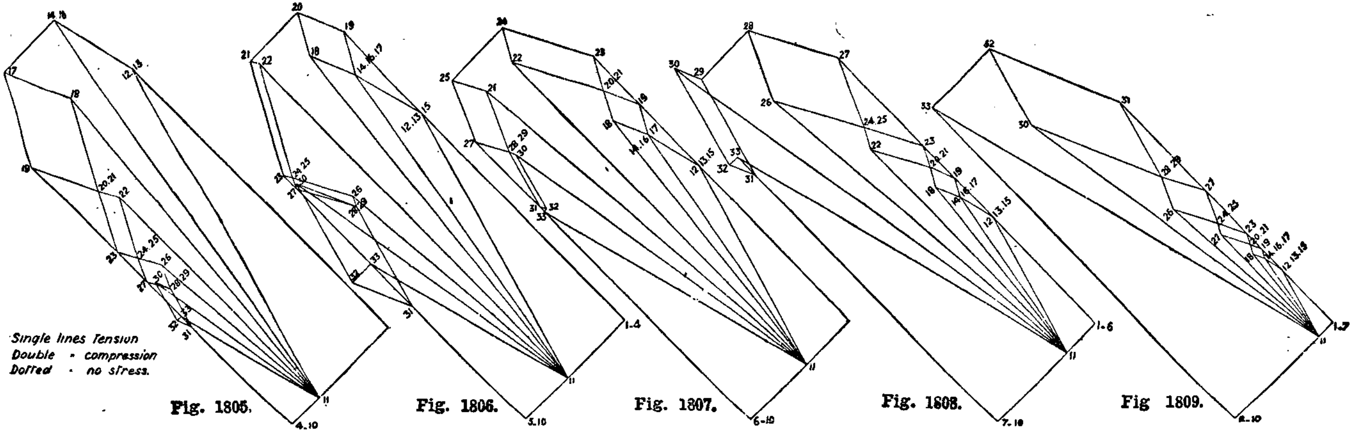
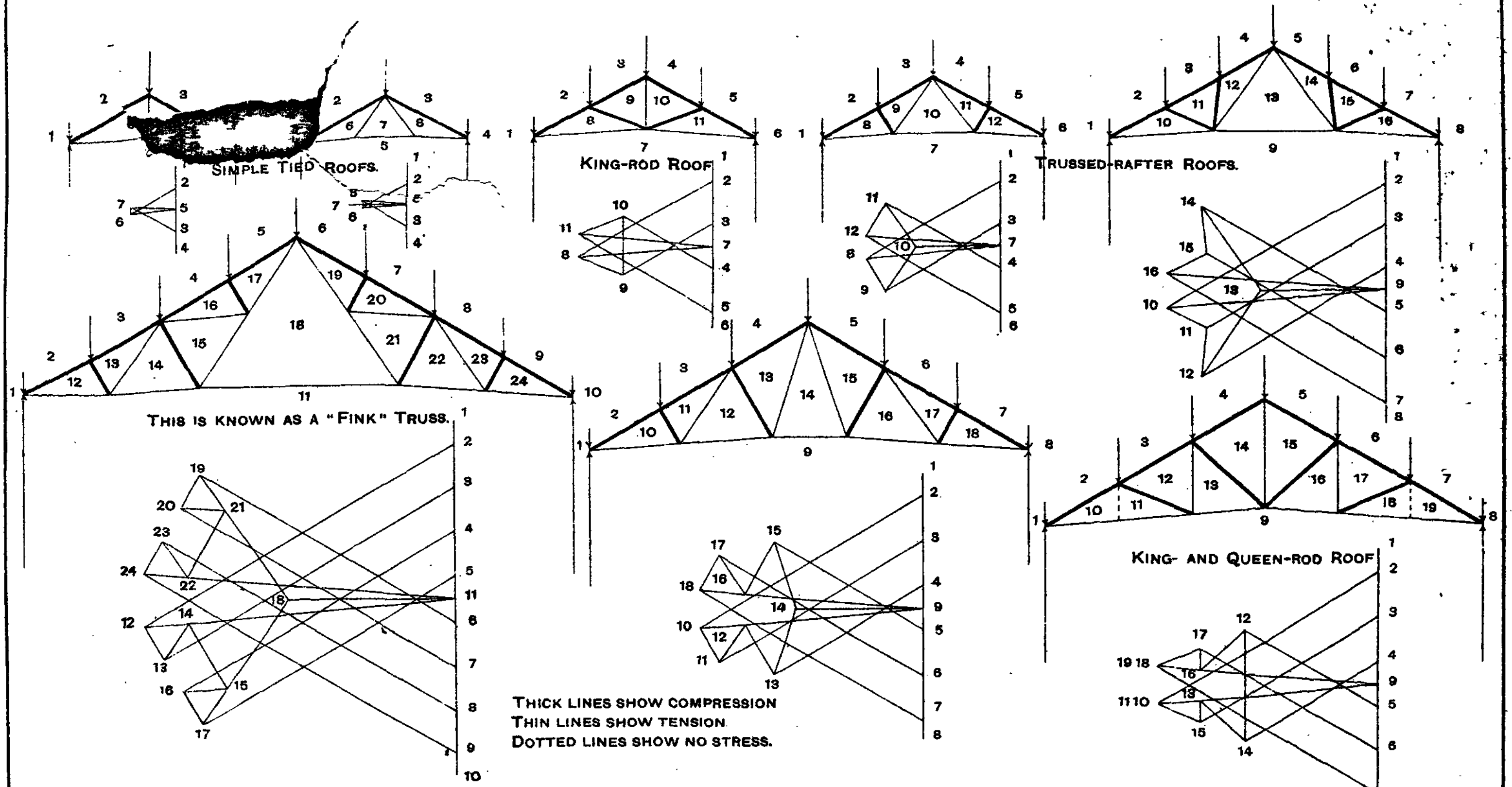

Single lines Tension
Double " compression
Dotted " no stress.
4.10
1.4
3.10
6-10
1.6
7.10
1.7
8.10
11

Fig. 1805. **Fig. 1806.** **Fig. 1807.** **Fig. 1808.** **Fig 1809.**

Figs. 1801 to 1809.—Diagrams showing Stresses in Inclined Fish-bellied Girder.


Simple Tied Roofs.
King-rod Roof
Trussed-rafter Roofs.
This is known as a "Fink" Truss.
King- and Queen-rod Roof
Thick lines show compression
Thin lines show tension.
Dotted lines show no stress.

Weight of Truss in lbs.

$$\frac{\text{span ft.}^2 \times \text{distance apart ft.}}{24}$$

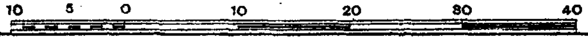
$$\text{Distance apart} = \frac{\text{span ft.}}{0.03\ \text{span} + 2}$$

1. Bay in rafter suitable for roofs up to 15-ft. span.
2. Bays " " 15—20 "
3. " " " 25—50 "
4. " " " 40—75 "
5. " " " 60—100 "

Total load on roofs.

½ cwt. per ft. sup. of actual surface is sufficient to allow for all contingencies.

Scale of Feet.


10
5
0
10
20
30
40

STRESS DIAGRAMS FOR IRON ROOF TRUSSES.

right through; and the same figures with the minus sign before them will be put on the other bar of each pair—that is, the one inclined downwards towards the right abutment. Then all the figures + and − are to be repeated from the right abutment towards the left, namely + for the members inclined downwards towards the abutment from which the start is made, and − for the other member of each pair. The values of the stresses by calculation are inserted for comparison on the extreme right of the table. It will be seen that they are somewhat in excess of the maximum stresses found graphically. No doubt the two sets ought to agree, and the writer is unable to say why they do not. A uniformly distributed dead load will produce a shear stress in the web corresponding to ordinates to the straight lines A E and E D, while the increased shear stress produced by a rolling load is shown by the space between these and the semi-parabolas as A B and C D, showing an important reason for giving due consideration to a rolling load; but the change of stress produced in the bars as the load passes along is also an essential consideration. Large structures are not simply magnified representations of smaller ones. Generally speaking, in order to obtain the

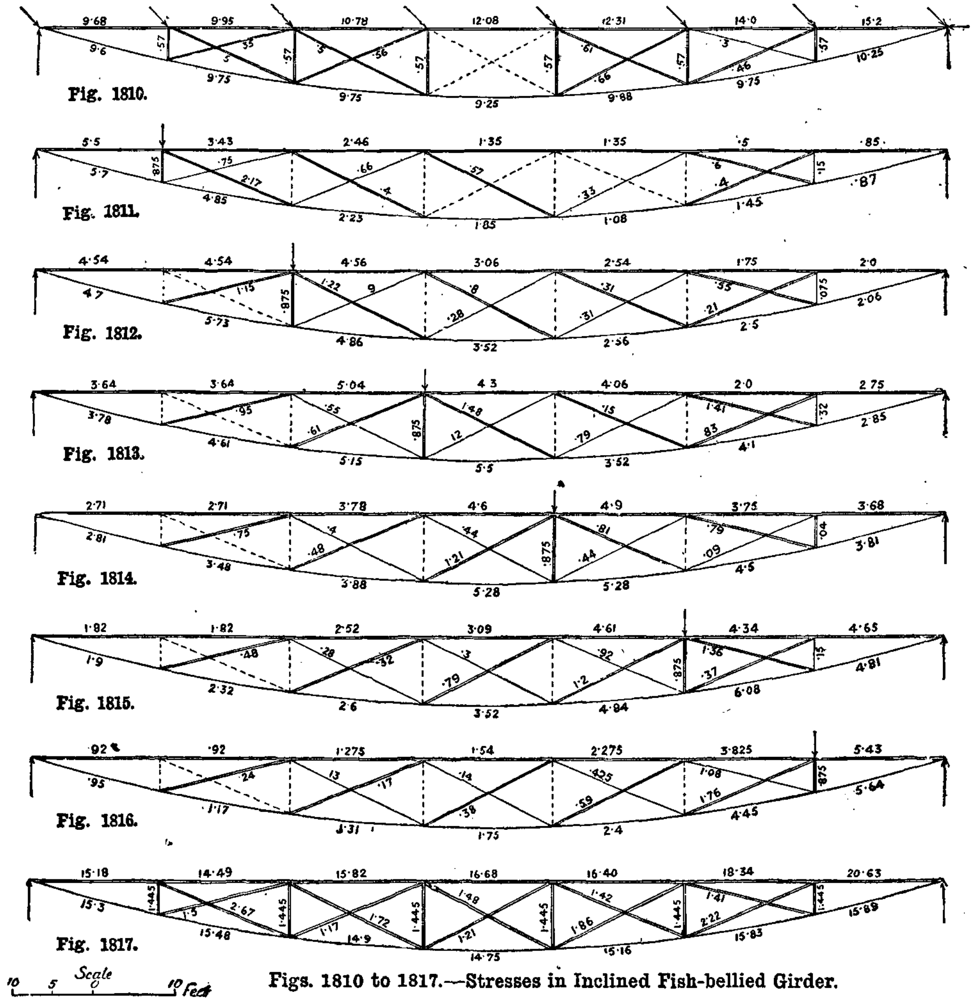


Fig. 1810.

Fig. 1811.

Fig. 1812.

Fig. 1813.

Fig. 1814.

Fig. 1815.

Fig. 1816.

Fig. 1817.

9·68
9·95
10·78
12·08
12·31
14·0
15·2
9·6
9·75
9·75
9·25
9·88
9·75
10·25
5·5
3·43
2·46
1·35
1·35
·85
5·7
4·85
2·23
1·85
1·08
1·45
·87
4·54
4·54
4·56
3·06
2·54
1·75
2·0
4·7
5·73
4·86
3·52
2·56
2·5
2·06
3·64
3·64
5·04
4·3
4·06
2·0
2·75
3·78
4·61
5·15
5·5
3·52
4·1
2·85
2·71
2·71
3·78
4·6
4·9
3·75
3·68
2·81
3·48
3·88
5·28
5·28
4·5
3·81
1·82
1·82
2·52
3·09
4·61
4·34
4·65
1·9
2·32
2·6
3·52
4·94
6·08
4·81
·92
·92
1·275
1·54
2·275
3·825
5·43
·95
1·17
1·31
1·75
2·4
4·45
5·64
15·18
14·49
15·82
16·68
16·40
18·34
20·63
15·3
15·48
14·9
14·75
15·16
15·83
15·89
Scale
10
5
0
10 Feet

Figs. 1810 to 1817.—Stresses in Inclined Fish-bellied Girder.

requisite material, the parts, such as rivets and plates, must be multiplied in number rather than in size.

Stresses in Inclined Fish-Bellied Girder.

It is required to find the stresses on the various members of the inclined girder shown in Fig. 1801. The moving load is reckoned at $2\frac{1}{2}$ tons, the dead load at 5 tons, and the wind pressure at 3 tons. A moving load on an inclined girder acts by gravity vertically downwards, and this force is resolved into two others as shown in Fig. 1802, where W is the load, P the pull on the chain, or power to pull the load up or absorbed in friction in running down, and T the transverse thrust on the girder. The dead load and the wind pressure may be equally divided over the seven bays, and applied where the perpendiculars meet the chord. The moving load must be taken consecutively on all the apices, one at a time. Half of each apex load may be assumed to pass down the perpendicular strut. The frame diagram with external distributed forces will then be as shown in Fig. 1801, and the stress diagram as in Fig. 1803. The stresses scaled off will be as shown on Fig. 1810. Now, keeping the same numbering as in Fig. 1803, it will be necessary to make stress diagrams for the consecutive positions of the moving load as shown in Figs. 1804 to 1809, and scale off the stresses and mark them down as shown in the corresponding Figs. 1811 to 1816. Next, taking a new frame diagram (Fig. 1817), the maximum stresses on each member must be marked, namely, the value on Fig. 1804, added to the greatest value of the same member on any one of Figs. 1811 to 1816. Fig. 1817 will then show the values to use in designing the girder, but allowance must be made for the combination of the longitudinal and transverse stress as the moving load is passing from apex to apex.

STRUTS, STANCHIONS, AND ARCHED RIBS.

Calculating Size of Principal Rafter.

Gordon's Formula.—This formula is as follows:—$t = \dfrac{f}{1 + \dfrac{a\,l^2}{c\,d^2}}$, where l = length in inches; d = least diameter in inches; f = greatest intensity of stress in tons per square inch due to thrust and flexure when on the point of buckling (for wrought iron = 18, for mild steel = 26); t = average thrust in tons per square inch on section of strut; a = constant depending on conditions of ends (for both ends fixed = 1, for both ends pivoted = 4, for one end fixed and the other pivoted = 2·5, for one end fixed and the other free = 16); c = constant, depending on the shape of the cross section and the nature of the material, which for rectangular or cylindrical solid bars of wrought iron or mild steel = 2500, for cylindrical tubes of wrought iron or mild steel = 3500, for angle, T, channel, rolled joist, bars and distance pieces, hollow-square and built sections generally = 900. By this formula a 3-in. by 3-in. by ½-in. wrought-iron principal rafter, T-section, 9 ft. long, taken as one end fixed and the other pivoted, with a factor of safety of 4, and a least diameter of 2·95 in., as shown in Fig. 1818,

$$t = \frac{\frac{1}{4} \times 18}{1 + \frac{2{\cdot}5}{900}\left(\frac{9 \times 12}{2{\cdot}95}\right)^2} = \frac{4{\cdot}5}{1 \times \frac{1300}{360}} = 1 \text{ ton per}$$

square inch. Area $= \frac{1}{2}(3 + 2\frac{1}{2}) = 2{\cdot}75$ sq. in., and $1 \times 2{\cdot}75 = 2{\cdot}75$ tons safe load. For a load of 7·8 tons a trail must be made of a likely section, say 5 in by 4 in. by ½ in., with a least diameter of 3·95 in., as shown in Fig. 1819.

$$\text{Then } t = \frac{\frac{1}{4} \times 18}{1 + \frac{2{\cdot}5}{900}\left(\frac{9 \times 12}{3{\cdot}95}\right)^2} = \frac{4{\cdot}5}{1 + \frac{748}{360}} =$$

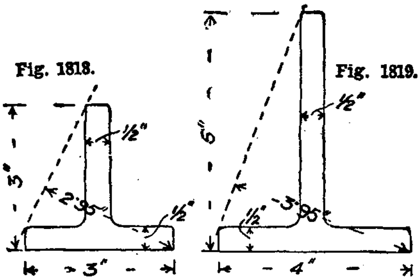
1·46 tons per square inch. Area $= \frac{1}{2}(4 + 4\frac{1}{2}) = 4{\cdot}25$ sq. in., and $1{\cdot}46 \times 4{\cdot}25 = 6{\cdot}22$ tons safe load, which is getting close, but not quite sufficient. The next size, say 5 in. by 5 in. by ½ in., will therefore be the proper section. Experience is a great help in designing, as less time is then likely to be wasted in calculating improbable sections.

30-ft. Mild Steel Built-up Stanchion.

Fidler's Formula.—In a given case it is required to find the safe load for a stanchion that is built up with four ⅝-in. by 6-in. by 6-in. angles (mild steel), all riveted together (the rivets being 3 in. apart); height 30 ft., with plate cap and base. A stanchion that is made of four angles riveted back to back is not an economical one, on account of the amount of riveting and the mass of the section being collected near the centre. The best formula for strength is Fidler's practical formula, for which the moment of inertia and radius of gyration must be found. In order to do this, the section is first drawn out as shown in Fig. 1820. The section may then be divided into three portions, as B C D (Fig. 1821), and the areas of these portions found, no rivet holes being deducted, as the column is in compression. These areas are then set off to a linear scale, as shown in Fig. 1822, and vectors are drawn to a pole O, O x being perpendicular to the line of areas and equal to half the length of this line. By drawing concurrent lines parallel to these vectors, the inertia area will

Fig. 1818.

Fig. 1819.



Figs. 1818 and 1819.—Calculating Size of Principal Rafter.

be found as shown in Fig. 1823 = 5·6 sq. in. Then the moment of inertia I = A a, where A = area of whole section and a = inertia area, therefore I = A a = 30 × 5·6 = 168 in square inch units. For the radius of gyration,

$$r = \sqrt{\frac{\text{I}}{\text{A}}} = \sqrt{\frac{168}{{\cdot}30}} = \sqrt{5{\cdot}6} = 2{\cdot}36 \text{ in.}$$

Then by Fidler's formula p = load in pounds to produce stress f, f = ultimate compressive stress in pounds per square inch = for wrought iron 36,000 lb. per square inch, for cast iron 80,000 lb., for hard steel 70,000 lb., for mild steel 48,000 lb., R = resilient force of ideal column in pounds per square inch, L = length of column in inches, l = for fixed ends, $\frac{6}{10}$ L, r = radius of gyration in inches measured in plane of easiest flexure, E = modulus of direct elasticity of material = for wrought iron 26,000,000, for cast iron 14,000,000, for mild steel 29,000,000. Maximum p or B. W. of ideal column = R = E $\pi^2 \times \left(\frac{r}{l}\right)^2$; minimum p or B. W. of practical column =

$$\frac{f + \text{R} - \sqrt{(f + \text{R}) - 2{\cdot}4 f \text{R}}}{1{\cdot}2}$$

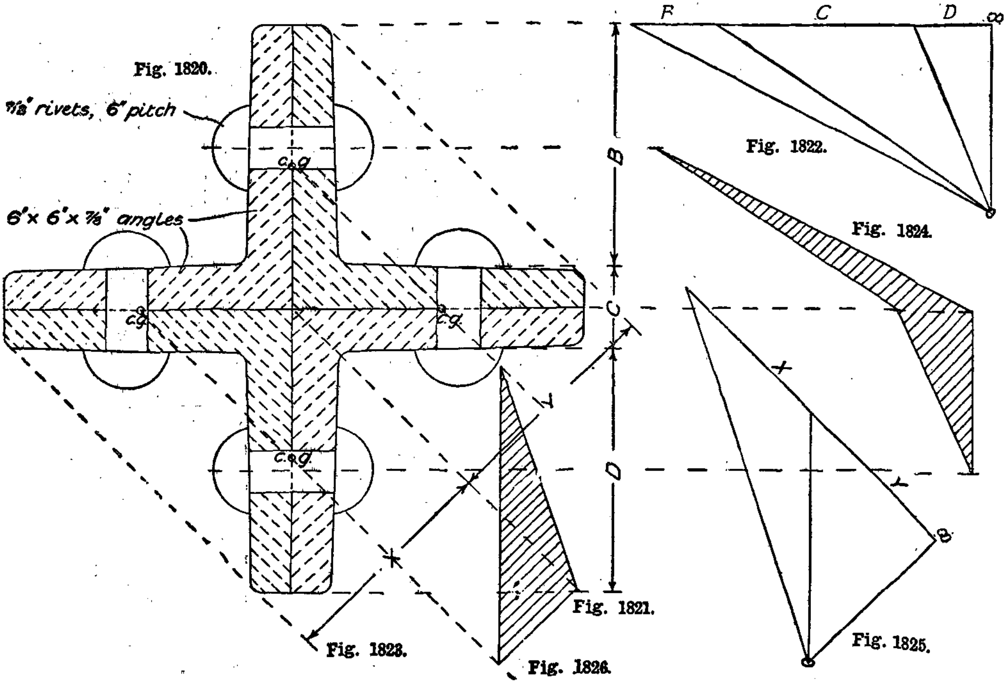


Fig. 1820. Fig. 1821. Fig. 1822. Fig. 1823. Fig. 1824. Fig. 1825. Fig. 1826.

Figs. 1820 to 1826.—Finding Safe Load for Built-up Stanchion.

(NOTE.—The tensile and compressive strength of mild steel may have almost any value between that of hard steel and that of good wrought iron, and the stronger the steel the higher will be the value of f in the formula; 60,000 lb. is a common average.) Then R = E $\pi^2 \times \left(\frac{r}{l}\right) = 29000000 \times 3{\cdot}1416^2 \times \left(\frac{2{\cdot}36}{\frac{6}{10} \times 30 \times 12}\right)^2$ = 35090 lb. per square inch ideal breaking weight, whence p =

$$\frac{60000 + 35090 - \sqrt{(60000 + 35090)^2 - 2{\cdot}4 \times 60000 \times 35090}}{1{\cdot}2}$$

= 26625 lb. per square inch practical breaking weight. Factor of safety = 4 + ·05 $\left(\frac{l}{d}\right)$ = 4 + ·05 $\left(\frac{30 \times 12}{9}\right)$ = 6. Then $\frac{26625}{6}$ = say 4440 lb. per square inch safe load, and $\frac{4440 \times 30}{2240}$ = say 60 tons safe dead load if the whole of it is a purely axial load, and the bending is possible only in the direction of the joints. If, however, the bending may take place in any direction, it will naturally choose the weakest (namely, diagonally across the angles), and the strength will be found as follows. By dividing the section into portions X Y at right angles to the least diameter, and proceeding as before, a will be found to = 4·84 sq. in., as in Figs. 1824, 1825, and 1826, whence $\text{I} = \text{A}$, $a = 30 \times 4{\cdot}84 = 145{\cdot}2$, and $r = \sqrt{\frac{\text{I}}{\text{A}}} = \sqrt{\frac{145{\cdot}2}{30}} = 2{\cdot}2$ in.

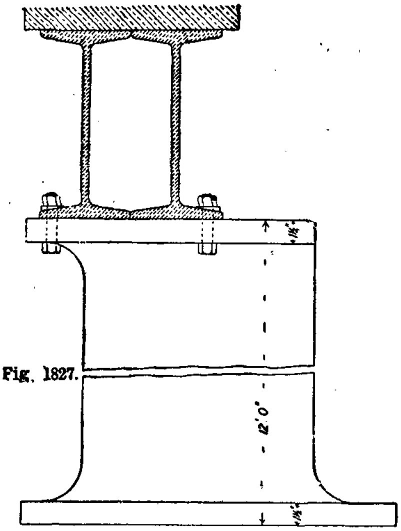


Fig. 1827.

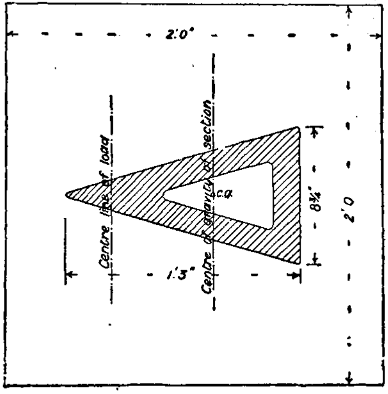


Fig. 1828.

Figs. 1827 and 1828.—Elevation and Horizontal Section of Three-sided Column.

Then $\text{R} = \text{E}\,\pi^2 \times \left(\frac{r}{l}\right)^2 = 29000000 \times 3{\cdot}1416^2 \times \left(\frac{2{\cdot}2}{\frac{6}{10} \times 30 \times 12}\right)^2 = 29000$ lb. per square inch ideal breaking weight, whence $p =$

$$\frac{60000 + 29000 - \sqrt{(60000 + 29000)^2 - 2{\cdot}4 \times 60000 \times 29000}}{1{\cdot}2}$$

= 23175 lb. per square inch practical breaking weight. Then allowing the same factor ot safety 6, $\frac{23175}{6} = 3862$ lb. per square inch safe load, and $\frac{3862 \times 30}{2240}$ = say 51 tons safe dead load. The weight of this stanchion will be about 1·4 tons, exclusive of top and bottom flanges, which at £14 per ton is approximately £19 12s. If a simple rolled joist had been used, a "Differdange" section, 45 B 121 lb. per foot run, would have been sufficient, weighing 1·6 tons, which at £10 per ton would amount to £16, showing a saving of 18 per cent.

Strength of Three-sided Column.

There are occasions when a stanchion of peculiar shape may be required to meet certain conditions; for instance, a three-sided column, as shown in Figs. 1827 and 1828. The load consists of a 14-in. wall supported on the girders shown, bolted to the head of the column seen in Fig. 1827, a sectional plan being presented by Fig. 1828. It will be necessary, before the supporting power can be arrived at, to cut out and suspend a section of the column to find the centre of gravity through which the neutral axis will pass, and to find the area of section by planimeter. The centre of loading is at a distance of 3 in. from the point, the neutral axis 9½ in. from the point, and the area of section 49·92, say 50 sq. in. Another drawing of the section to an enlarged scale must now be made, and the inertia areas constructed as in Fig. 1829, the areas being obtained by planimeter. They must then be cut out and suspended to find the centre of gravity of each. The moment of inertia will then be the distance from neutral axis to edge × the inertia area × distance from neutral axis to centre of gravity of inertia area + the same for other side of neutral axis = 5·5 × 61·32 × 4·125 + 9·5 × 8·96

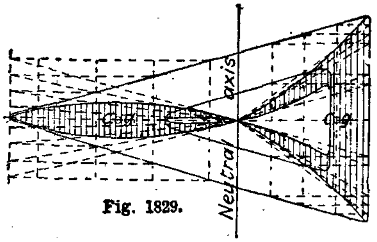
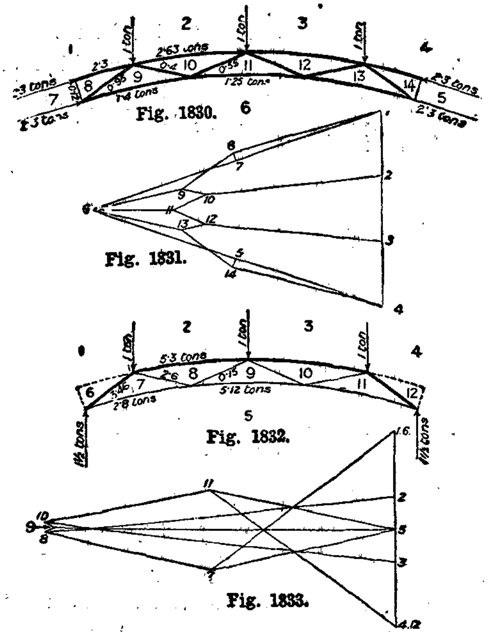


Fig. 1829.

Fig. 1829.—Finding Strength of Three-sided Column.

× 5·125 = 370·26 + 436·24 = 806·5 in inch units. Then the maximum stress $S = \frac{W}{A}\left(1 + d\frac{DA}{I}\right)$, where W = total non-axial load in tons, d = distance of centre of pressure of non-axial load from neutral axis of section, A = area of cross section in square inches, S = maximum stress in tons per square inch, D = distance of point of maximum stress from neutral axis in inches, I = moment of inertia of section. For present section, cast iron being capable of bearing a maximum working load of 7 tons per sq. in. in compression, $7 = \frac{W}{50}\left(1 + 6·5\,\frac{9·5 \times 50}{806·5}\right)$, or $7 = \frac{W}{50}(1 + 3·8)$, or $W = \frac{7 \times 50}{4·8} = 72·9$, say 73 tons total load irrespective of height of column. There is some doubt as to the next step in the procedure, as it is difficult to say how many diameters high the column is. Assuming the least diameter to be represented by the width across the centre line of loading, 2 in., the ratio of height to diameter will be $\frac{12 \times 12}{2} = 72$. Then by Gordon's formula, as given on p. 483, $p = \frac{f}{1 + \frac{a}{c}\left(\frac{l}{d}\right)^2}$ $= \frac{7}{1 + \frac{1}{1000}(72)^2} = \frac{7}{6·184} = 1·132$ tons per sq. in., instead of 7 as allowed for short section. Then as 7 tons per sq. in. is to 1·132 tons per sq. in., so is 73 tons total load to $\frac{1·132 \times 73}{7}$ = 11·8 tons total safe load as column 12 ft. high. This seems small, but is due to the centre line of load being so near the point of the section. There is much doubt as to what ought to be taken as least diameter, as although the column as a whole would not bend sideways, the thin edge would be very liable to buckle under compression.



Figs. 1830 to 1833.—Finding Strength of Curved Steel Joist.

Strength of Curved Steel Joist.

In finding the strength of a curved rolled joist used as an arch, a point of primary importance is to know the condition of the abutments. If the abutments are absolutely rigid and remain so, the rolled joist will be an arch rib subject to compression only, when loaded uniformly; but if the abutments yield, the rolled joist will be in the condition of an ordinary beam, compressed above and extended below, with the further disadvantage that the material may have suffered some damage in the bending. If a bent lattice girder be taken, a simple means of investigating the stresses is by reciprocal diagrams. Fig. 1830 shows the girder as an arch with rigid abutments, Fig. 1831 the corresponding stress diagram; Fig. 1832 shows the girder with the abutments yielding and causing a greatly increased stress on every part, as may be measured from the stress diagram (Fig. 1833). Except for a slight concentration of stress at various parts, this lattice girder shows precisely what takes place with a solid web girder, and a similar method may be adopted for ascertaining the stresses under irregular loading.

Strength of Curved Strut.

It is required to find the strength of a bent strut; for example, a 5-in. by 5-in. by 24-lb. rolled steel joist, in the circumstances shown. First let the thrust be along the chord of the arc, as shown in the curved strut A B (Fig. 1834). Then for the reciprocal diagram draw the chords A C, C B, and the vertical force line as shown, and number the spaces. Next draw 2-3 (Fig. 1835) equal to the thrust x, and draw 2-4 3-4, 4-1 2-1, and 1-5 4-5. Then 1-2 will be the external load that maintains equilibrium, or will represent the transverse stress on the strut resisted by its own stiffness. In the case of a 5-in. by 5-in. by 24-lb. rolled steel joist, the moment of inertia $\text{I} = 29{\cdot}55$ in.-lb., the moment of resistance (or, more correctly, the modulus of section) $\text{Z} = 11{\cdot}82$ sq.-in. units. Then as a beam supported at the ends with a central load $\frac{\text{W}\,l}{4} = \text{Z C}$, or $\text{W} = \frac{4\ \text{Z C}}{l} = \frac{4 \times 11{\cdot}82 \times 6}{10 \times 12} = 2{\cdot}364$ tons. Then rise : $\frac{1}{2}$ span : : $\frac{1}{2}$ load : thrust, whence the thrust $= \frac{\frac{1}{2}\text{ span} \times \frac{1}{2}\text{ load}}{\text{rise}} = \frac{5 \times 1{\cdot}182}{\cdot 5} = 11{\cdot}82$ tons safe load. Using these figures, the reciprocal diagram will be found to close, the case being peculiar because the reciprocal diagram must be sketched in order that the values may be reasoned out, and then the values must be used in order to obtain the reciprocal diagram to scale. The theory of this method is, that the same section used as a beam supported at the ends would bear a safe central load of 2·364 tons, this being what may be called the value of the cross section for the given length. Then supposing the strut to be cut in halves and jointed by a pivot in the centre, with this load acting transversely to the pivot, the end thrust of 11·82 tons would balance the load ; that is, the whole section being worth 2·364 tons, that load should be applied, when the section is cut through so that it has no transverse strength left, to find the equivalent end thrust. The problem may possibly be attacked on the principle of the bending moment produced by a non-axial load. First, by Gordon's formula (see p. 483) find the maximum safe stress due to the strut taken as a simple column, $t = \frac{f}{1 + \frac{a\,l^2}{c\,d^2}}$ where l and d are length and diameter in inches, f is the compressive strength in tons per square inch, = 26 for mild steel, a constant = 1 for both ends fixed, c constant for shape of section = 900 for rolled joist, t crippling thrust tons per square inch with central load. Then $t = \frac{26}{1 + \frac{1 \times 120^2}{900 \times 5^2}} = 15{\cdot}85$ tons per square inch. For a strut twenty-four diameters long the factor of safety should not be less than 5 ; the working stress as a straight column will thus be $\frac{15{\cdot}85}{5} = 3{\cdot}17$ tons per square inch. For the effect of the bending moment due to a thrust along the chord of the arc, the formula is $\text{P} = \frac{\text{W}}{\text{A}} + \frac{\text{M}}{\text{Z}}$ where P is the working stress just found = 3·17, W the load (which has to be found) that the strut can carry in tons, A = area of strut in square inches = 7·04, Z modulus of section = 11·82, M bending moment = W × 6 in. Then $3{\cdot}17 = \frac{\text{W}}{7{\cdot}04} + \frac{6\ \text{W}}{11{\cdot}82}$, or $\text{W} = \frac{3{\cdot}17 \times 7{\cdot}04 \times 11{\cdot}82}{6 \times 7{\cdot}04 + 11{\cdot}82} = 4{\cdot}885$ tons safe load. These solutions of the problem are put forward tentatively ; they cannot both be right. It does not appear that any solution of the problem has ever been published, although the case would appear to be one that frequently recurs.

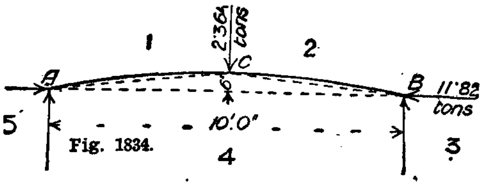


Fig. 1834.

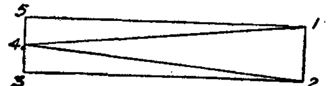


Fig. 1835.

Figs. 1834 and 1835.—Finding Strength of Curved Strut.

Stresses in a Concrete and Steel Dome.

In the following case of an arched dome about 53 ft. 6 in. in diameter at the base, and supported by twenty-four steel ribs, with concrete filling, if a section of the dome be taken as a linear arch, with the rib on one side subject to wind pressure and structural load and the rib on the other side subject to structural load only, and sufficient sectional area be provided, there is little doubt that the structure would be safe. Taking Figs. 1836 and 1837 as the assumed section of one bay, top and bottom, the structural weight supported by each rib will be approximately as marked on one bay in the half elevation (Fig. 1838), and the wind pressure as marked on the

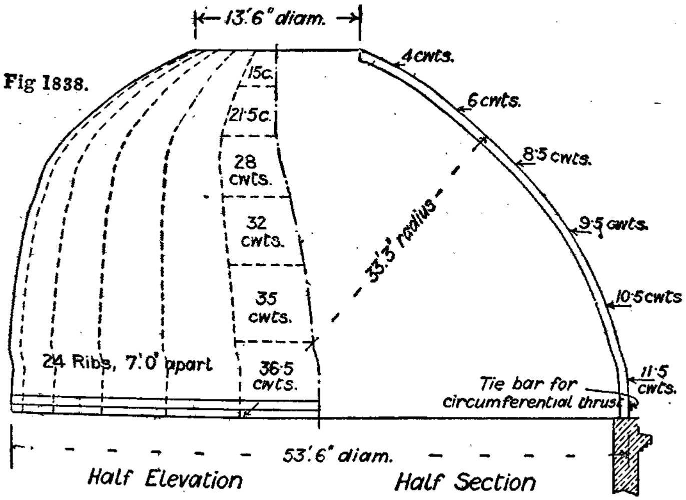


Fig 1838.
13'6" diam.
4 cwts.
6 cwts.
8·5 cwts.
9·5 cwts.
10·5 cwts
11·5 cwts.
15c.
21·5c.
28 cwts.
32 cwts.
35 cwts.
36·5 cwts.
33'3" radius
24 Ribs, 7'0" apart
Tie bar for circumferential thrust
53'6" diam.
Half Elevation
Half Section

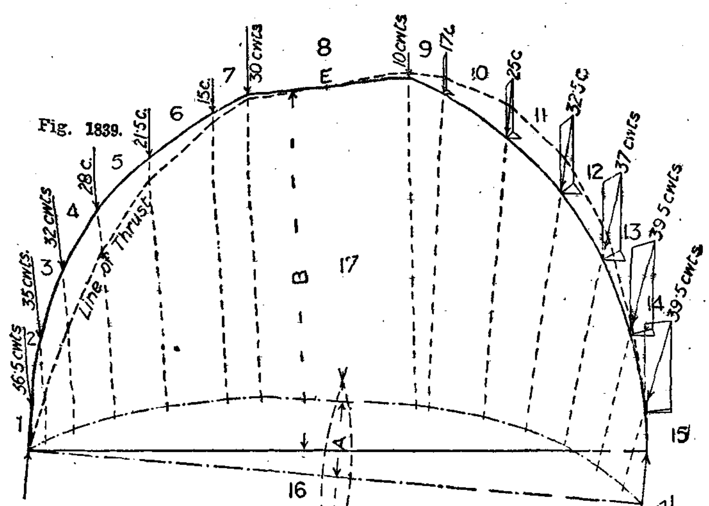


Fig. 1839.
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
E
B
A
56·5 cwts
35 cwts.
32 cwts
28 c.
21·5 c.
15 c.
30 cwts
10 cwts
17c.
25c.
32·5c.
37 cwts
39·5 cwts
39·5 cwts
Line of Thrust

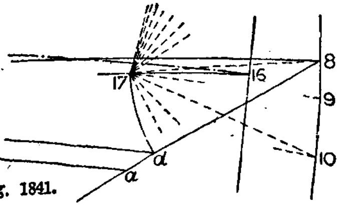


Fig. 1841.

Enlargement from Fig. 1840
8
9
10
16
17
11
d
a

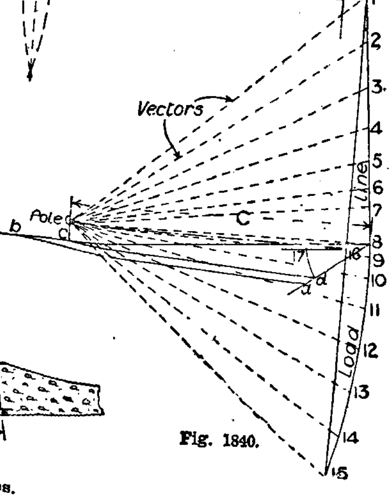


Fig. 1840.
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Vectors
Pole
c
C
b
d
a
Load Line

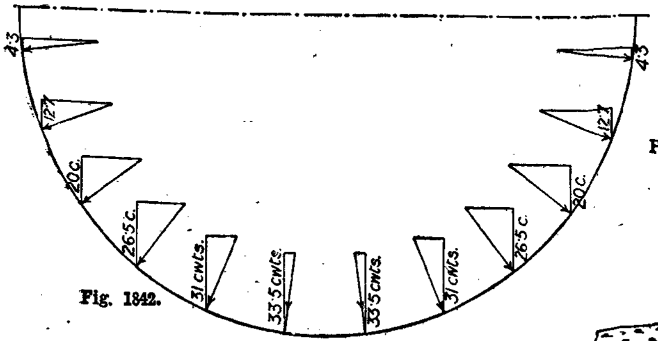


Fig. 1842.
4·3
12·7
20c.
26·5c.
31 cwts.
33·5 cwts.
33·5 cwts.
31 cwts.
26·5 c.
20c.
12·7
4·3

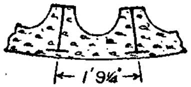


Fig. 1837.

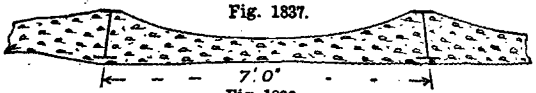


Fig. 1836.

Figs. 1836 to 1842.—Concrete and Steel Dome: Details and Stresses.

half section, allowing 28 lb. per square foot horizontally. There will probably be some lantern, or other weight, on top of the dome, and this may be assumed at 1 ton on each rib. The frame diagram will then be as in Fig. 1839, where the wind against the lantern is assumed to halve the load on one side and increase it on the other side by the same amount. The horizontal wind pressure has to be reduced by parallelogram of forces to find the normal, and this again combined with vertical load to find the resultant. To find the line of thrust, begin by numbering the spaces of the frame diagram, and then drawing the load line as in Fig. 1840; joining points 1 and 15 will give the direction of the reactions in the frame diagram. Next select any pole (Fig. 1840), and draw vectors. Parallel to these vectors, draw a funicular polygon, as shown by stroke-and-dot lines in Fig. 1839, and parallel to the closing line draw 0—16 (Fig. 1840). In order to get the line of thrust to pass through the centre of the crown of the dome E, proceed as follows:—From point 8 (Fig. 1840) draw horizontal line, making 8c = C, and, continuing it, make 8b = B (Fig. 1839). From point 8 also set out a line at any angle with the horizontal (say 30°), making 8a = A (Fig. 1839). Next join B to A, and parallel to this draw a line from c, cutting 8a in d. Now, with point 8 as centre, and 8d radius, swing an arc to meet a horizontal line from point 16. This will give the position of a new pole point 17 (see also Figs. 1840 and 1841), from which the funicular polygon forming the line of thrust may be drawn as shown by strong dotted line (Fig.

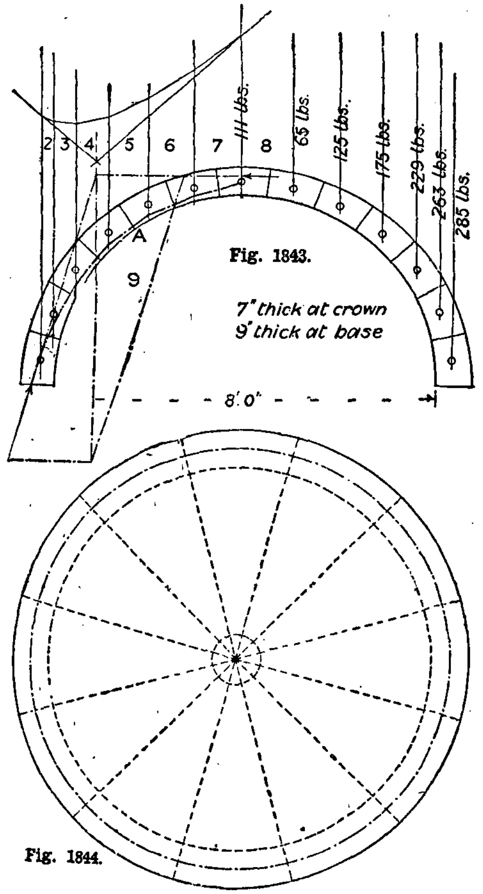


Fig. 1843.

Fig. 1844.

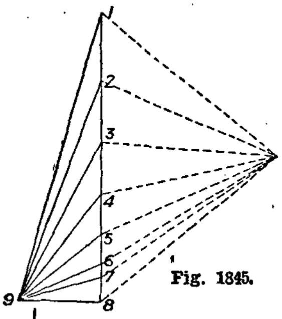


Fig. 1845.

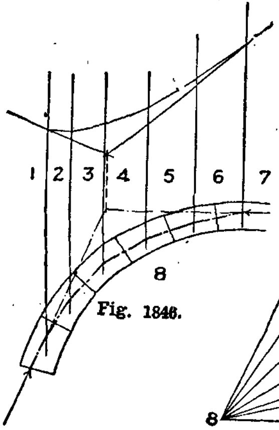


Fig. 1846.

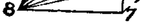


Fig. 1847.

Figs. 1843 to 1847.—Diagrams showing Stresses in Masonry Dome.

1839). A portion of Fig. 1840 is enlarged (Fig. 1841), in order to show more clearly the method of working. Fig. 1842 is a plan of half the dome, showing the outward thrust of each rib at the base, namely, 16—17 (Fig. 1840), with the effect at right angles to a diameter, giving a total of 256 cwt., tending to separate the curb into two parts, and therefore a tension of $\frac{1}{2} \times 256 = 128$ cwt. on the section of one side. At 5 tons per square inch tensile stress, a section of $\frac{128}{20 \times 5} = 1{\cdot}3$ sq. in. will be sufficient for the curb, say a bar 3 in. by ½ in. as a minimum; but, of course, something more than this would be desirable to allow for corrosion and joints. It might be bedded in the concrete at the back of the gutter. For the section of the ribs, the writer can only offer an approximate suggestion. The loads have been taken out for 12-in. by 6-in. by 54-lb. rolled steel joists, but calculation shows this to be in excess, and 10-in. by 5-in. by 35-lb. rolled steel joists would be more suitable. The effect of the line of thrust departing from the centre line of the rib is to cause a bending moment equal to the thrust multiplied by the distance between thrust line and rib. Adopting this would lead us to the formula $\frac{\text{W}}{\text{A}} \pm \frac{\text{M}}{\text{Z}}$ for the stress on the two edges of rib, where W = thrust in tons, A = area of section in square inches, M = bending moment in inch tons, Z = modulus of section in inch units (wrongly called moment of resistance in square inches). Then for the 10-in. by 5-in. by 35-lb. rolled steel joist (G 10 in Dorman, Long & Co.'s catalogue) $\frac{\text{W}}{\text{A}} \pm \frac{\text{M}}{\text{Z}} = \frac{5{\cdot}65}{10{\cdot}28} \pm \frac{5{\cdot}65 \times 27}{31{\cdot}99} = \begin{array}{l}\pm\, 5{\cdot}318\\ -\, 4{\cdot}219\end{array}$, or say 5·3 tons per square inch compression, and 4·2 tons per square inch tension. The 5·65 in this calculation is the thrust 4—17 scaled in cwt., and divided by 20 for tons; and 27 is the distance in inches between full line and dotted line across 3—17. There should be a connecting band round the ribs half-way up the dome, and a strong interior ring at the top.

Stability of a Masonry Dome.

The section of a proposed dome is shown in Fig. 1843, where the inside is hemispherical, but the outside, owing to the thickening towards the base, is rather less than a hemisphere. In order to bring this matter within the range of ordinary solutions, divide the plan (Fig. 1844) into, say, twelve equal parts, so that the opposite ones can be dealt with as a simple arch. Then divide up the section into assumed voussoirs, as shown in Fig. 1843, and from plan and section calculate the weight of each. These weights are shown at the right-hand side of the section, assuming the material to weigh 160 lb. per cubic foot. The figures at the top include the proposed additional load of 10 cwt. placed on the centre, one-twelfth of which is taken as forming part of the load to be borne by each of the twelve segments. Then, marking the centres of gravity of voussoirs on the left-hand side, drawing force lines, and numbering spaces, draw the load line (Fig. 1845), add the vectors, and draw the funicular polygon in Fig. 1843 to give the mean centre of effort of the loads through which a vertical line is drawn. From the outer edge of the middle third at the crown draw a horizontal line of thrust to meet the vertical just drawn, and from the outer edge of the middle third at base draw a thrust line to the intersection just found. These are the directions of the mean force lines acting in the structure, and the parallelogram may be completed by drawing a line in Fig. 1845 from point 1 parallel to the inclined line described above, and cutting it off at 9 by a horizontal line from point 8. Now, by transferring 1—9 and 8—9 to Fig. 1843, the parallelogram will be completed as shown. Draw lines in Fig. 1845 from each of the points on the load line to point 9, and the line of thrust may then be drawn on Fig. 1843 parallel to these, beginning at the base. It will be seen that the thrust comes very near the intrados at A. The distance scales ·07 ft., and the thrust at that joint scaled from Fig. 1845 is 480 lb. Then by the usual formula $\frac{2}{3}\,\frac{\text{W}}{d} = \frac{2}{3} \times \frac{480}{{\cdot}07} = 4571{\cdot}4$ lb., say 2·04 tons per square foot maximum compression. This is within the limit of the working stress; but with a line of thrust so near the edge of the joint, the structure cannot be considered satisfactory. Another inch added to the thickness all over would be very desirable. By omitting the bottom course of voussoirs, as in Figs. 1846 and 1847, the actual shape of the part left is unaltered, and yet the line of thrust is found to lie almost in the centre of the material.

STABILITY OF WALLS.

Resistance to Overturning.

A WALL of brickwork rises 10 ft. above the ground line; it is two bricks (18 in.) thick; it is exposed to the direct pressure of the wind at the rate of 25 lb. per square foot, and is found to have stood through the storm; it is required to know whether it would have been overthrown if it had been simply placed on the ground line, and had no support from the adhesion of the mortar. Taking 1 ft. run of the wall, the active force will be found thus:—Total pressure of wind on wall = 25 × 10 × 1 = 250 lb., acting at a leverage of 10 ÷ 2 = 5 ft., and producing a moment of 250 × 5 = 1,250 lb.-feet. The passive force or resistance will be found thus:—Weight of 1 cub. ft. of brickwork, 112 lb. Total weight of wall in 1-ft. run = 112 × 10 × 1·5 = 1,680 lb., acting with a leverage of 1·5 ÷ 2 = 0·75 ft., and producing a moment of 1,680 lb. × 0·75 = 1,260 lb.-feet, so that the wall will just stand, the ratio of stability being $\frac{1260}{1250} = 1\cdot008$. Algebraically, the investigation would stand thus:—

p = wind pressure pounds per square foot.
w = weight of brickwork pounds per cubic foot.
h = height of wall in feet.
t = thickness of wall in feet.
l = leverage of wind to toe of wall.
y = leverage of weight to ditto.
P = total pressure of wind on 1-ft. run of wall.
W = total weight of brickwork in ditto.

Then $P = ph$, and $W = wht$, and for equilibrium the moment Px should equal the moment Wy, while the ratio of stability in any given case will be—

$$\frac{whty}{phx} = \text{in this case } \frac{112 \times 10 \times 1\cdot5 \times 0\cdot75}{25 \times 10 \times 5} = 1\cdot008.$$

Or, let A B C D (Fig. 1848) be the section of the wall. At the centre of its height and from a line through its centre of gravity draw a horizontal line representing the total wind pressure P to any given scale, and a vertical line through the centre of gravity representing the weight W to the same scale. Complete the parallelogram, and the resultant falling just within the base shows that the wall will not overturn.

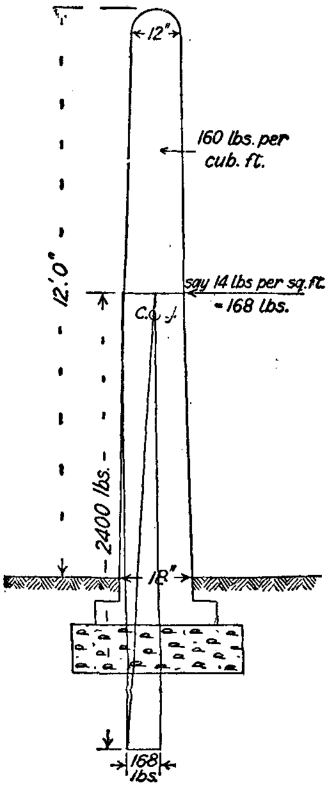


Fig. 1849.—Diagram showing Stability of Masonry Wall.

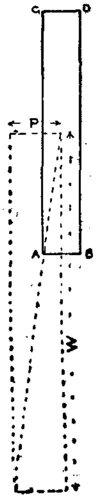


Fig. 1848. — Diagram showing Stability of Brick Wall.

Stability of Enclosure Wall.

Take the case of a very long enclosure wall, 12 ft. high, of fair coursed rubble masonry, averaging 8-in. courses, to be built on a sandy foundation in a damp and exposed situation near the sea. The wall being of rubble, and the blocks of stone varying in size, it may be built more economically by reducing the thickness towards the top. Counterforts are of much less value than buttresses, and unless particularly well bonded to the main wall are practically useless. As no figures are given for the maximum wind pressure, the proposed section will be subject to revision, but will be somewhere about what is shown in Fig. 1849. The whole of the work should be in hydraulic lime or cement, as a fat lime would never set firm under the conditions given. The illustration shows that a wind pressure of 14 lb. per ft. super. will throw the resultant over to $4\frac{1}{2}$ in. within the outer edge at the ground line. The load, or weight of wall per ft. run, is 2,400 lb.; therefore the maximum pressure at the outer edge will be $\frac{2}{3} \times \frac{W}{d} = \frac{2}{3} \times \frac{2400}{2240} \times \frac{12}{4\frac{1}{2}}$ $= 1·902$, say 2 tons per square foot, which ought to be safe.

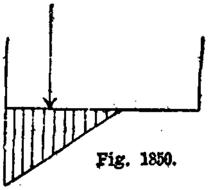


Fig. 1850.

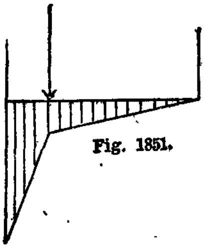


Fig. 1851.

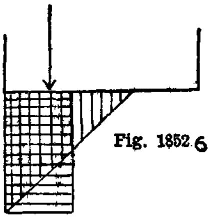


Fig. 1852.

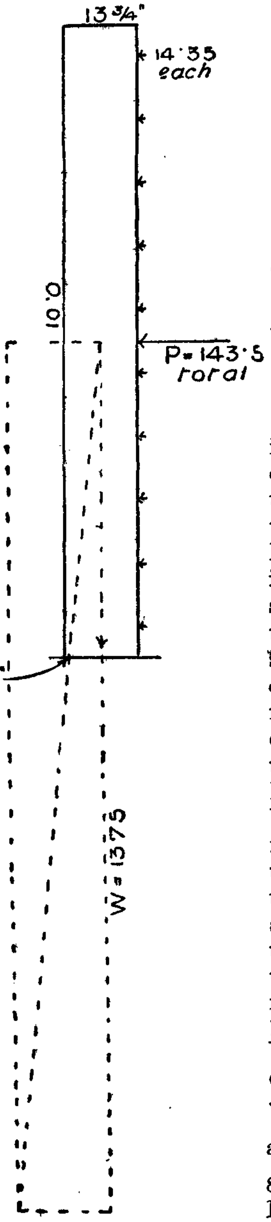


Fig. 1853.

Figs. 1850 to 1853.—Maximum Pressure on Base of Wall shown by Diagrams.

Maximum Pressure on Base of Wall.

A brick wall is 10 ft. high; it is $13\frac{3}{4}$ in. thick; it may be taken as resting, without adhesion, on a horizontal bed at the ground level, and on this it bears uniformly. Assuming its weight to be 120 lb. per cubic foot, and that its safe load (for crushing) is 8 tons per square foot, it is required to find what wind pressure per square foot it will safely bear, with the assumption of a maximum pressure of 8 tons per square foot. Before this can be answered correctly it will be necessary to determine the maximum pressure produced on the nearer edge of any section by a force acting out of the centre. The common theory is due to Professor Crofton, of the Military Academy, Woolwich, where the reactions against a surface, when the resultant is beyond the middle third, are as ordinates to a triangle which extends inwards twice the distance from the resultant to the outer edge (see Fig. 1850), while the author's investigations lead him to think that a more correct distribution would be by reactions shown in Fig. 1851. The wall weighs $\frac{13·75}{12} \times 10 \times 120 = 1375$ lb. per ft. run. On the ordinary hypothesis, if the reactions were evenly distributed, the wall would require a width of $\frac{1375 \times 12}{8 \times 2240} = ·9208$ in. loaded to 8 tons per square foot, as shown by the rectangle Fig. 1852. The base of this doubled will make a triangle of same height and equal area, representing the whole of the reactions, and one-third of the base will give the position of their centre of gravity, and therefore of the resultant, which will thus be $\frac{\cdot 9028 \times 2}{3} = \cdot 6139$ in. from the overturning edge. Then wind × its leverage = load × its leverage, or $\frac{\text{load} \times \text{its leverage}}{\text{wind leverage}}$ = wind force, $\frac{1375 \times \left(\frac{13\cdot 75}{2} - \cdot 6139\right)}{5 \times 12} =$ 143·5 lb., which spread over the height of 10 ft. = 14·35 lb. per square foot. This is checked graphically by Fig. 1853.

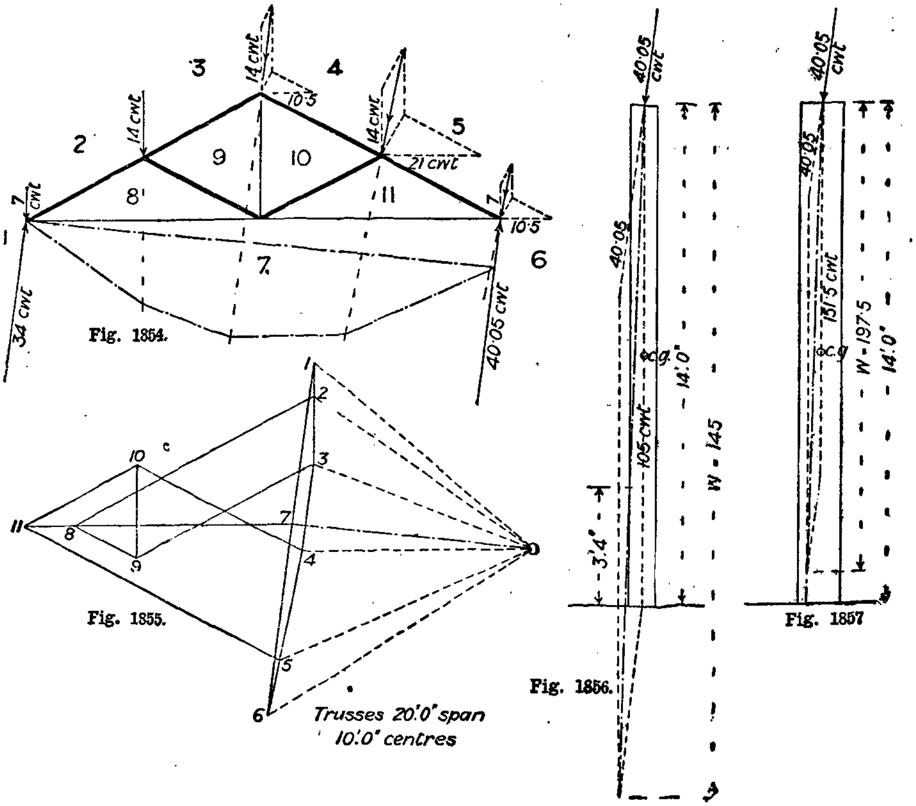


Fig. 1854.

Fig. 1855.

Trusses 20'0" span
10'0" centres

Fig. 1856.

Fig. 1857

Figs. 1854 to 1857.—Diagrams showing Stability of Wall against Roof Pressure.

Maximum and Minimum Pressure on Base of Wall.

The maximum pressure produced on the base of a wall, when the resultant of the forces is thrown out of the centre by wind pressure on one side, may be worked in two ways : (*a*) When tension is permissible on the inner edge ; (*b*) when no tension is possible, as in the case of a block simply standing on a flat base. (*a*) When tension is permissible on the inner edge or windward side of a structure, the formula is P max. $= p + \frac{M x}{I}$ where P max. = greatest intensity of stress per unit of surface (say tons per square foot), p = mean stress due to direct loading, $\frac{M x}{I}$ = additional stress due to bending moment, M = bending moment (say in ton-feet), x = distance from neutral axis to edge of section (in feet), I = moment of inertia (in feet units). Take the case of a brick boundary wall 10 ft. high, 18 in. thick, with tension permitted on the inner edge, subject to a wind pressure of 30 lb. per square foot. Then for 1 ft. run the moment of the wind will be $\frac{10 \times 30 \times 5}{2240} = \cdot 67$ ton-foot, and the moment of resistance will be $\frac{10 \times 1{\cdot}5 \times 120 \times d}{2240} = {\cdot}8\,d$, d being the distance from the centre of the base to the line of the resultant, whence $d = \frac{\cdot 67}{\cdot 8} = {\cdot}84$ ft. Next $p = \frac{10 \times 1{\cdot}5 \times 120}{2240} \div 1{\cdot}5 = {\cdot}5357$ tons per square foot due to the weight of the wall only. Then $\text{I} = \frac{b\,h^3}{12} = \frac{1 \times 1{\cdot}5^3}{12} = {\cdot}281$, $\frac{\text{M}\,x}{\text{I}} = \frac{\cdot 67 \times \cdot 75}{\cdot 281} = 1{\cdot}786$ tons per square foot, or together $\text{P} = {\cdot}5357 + 1{\cdot}786 = 2{\cdot}3217$ tons per square foot maximum pressure on leeward edge. When tension is allowed, the amount = P min. must be ascertained. When the resultant falls within the base P min. $= w\,h \left(\frac{6\,l}{t} - 2\right)$, and when the resultant falls outside the base P min. $= w\,h \left(\frac{6\,l}{t} + 2\right)$, where w = weight per cubic foot of wall, h = height, l = distance of resultant from the outer edge of the wall, t = thickness of wall. In the present case the resultant is outside the base, therefore

$$\text{P min.} = \frac{120}{2240} \times 10 \left\{ \frac{6\,(\cdot 84 - \cdot 75)}{1{\cdot}5} + 2 \right\} =$$

$1{\cdot}26$ tons per square foot. The working tension on mortar is, say, 15 lb. per square inch $= \frac{15 \times 144}{2240} = {\cdot}964$ ton per square foot, so that with a wind pressure of 30 lb. per square foot this wall would have the greater tendency to fail by tension on the inner edge. (*b*) Take the same wall, but assume the brickwork to be green (that is, just built), so that the mortar has no strength, and allow a wind pressure of 20 lb. per square foot. The moment due to the wind will be $\frac{10 \times 20 \times 5}{2240} = {\cdot}446$ ton-foot, and the moment of resistance will be $\cdot 8\,d$ as before, where d will now be $\frac{\cdot 446}{\cdot 8} = {\cdot}56$ ft. The distance of resultant from the outer edge will then be $\cdot 75 - \cdot 56 = \cdot 19$ ft. The vertical component of the resultant will be equal to the weight of the wall $= \frac{10 \times 1{\cdot}5 \times 120}{2240} = {\cdot}8$ ton, and the maximum intensity of pressure would be, say, $\frac{2}{3} \cdot \frac{\text{W}}{l} = \frac{2}{3} \cdot \frac{\cdot 8}{\cdot 19} \doteq 2{\cdot}8$ tons per square foot, as against 3·65 tons with a wind pressure of 30 lb. The calculation of chimney shafts is more complex, owing to the horizontal sections being hollow and from other causes.

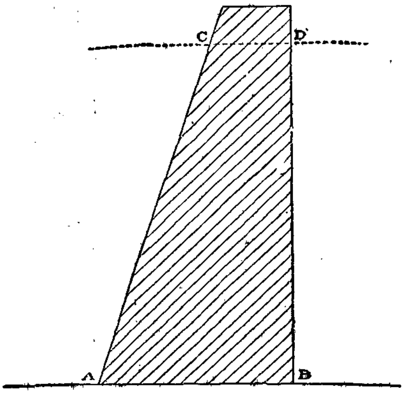


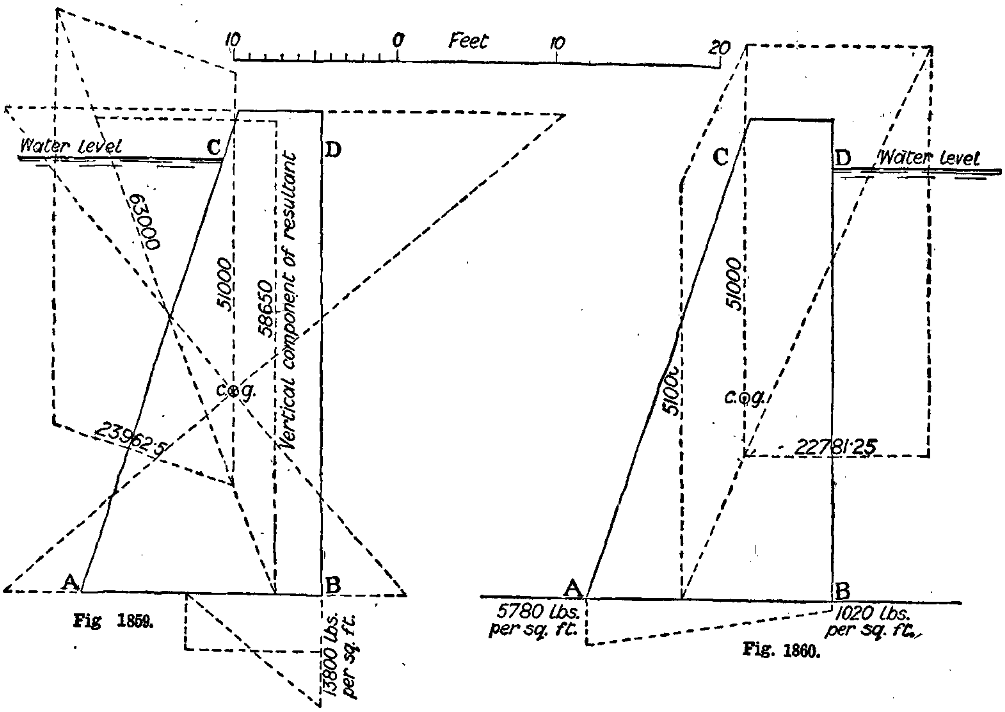
Fig. 1858.—Section of Battering Wall.

Stability of Wall against Roof Pressure.

A king-post roof truss, or any other framed truss coupled at the feet of the rafters, produces only a vertical load by its own weight. The effect of the wind on one side is to produce an increased load and an overturning effort against the walls. If the wind be taken as a vertical load, no side thrust can be produced; but this is not a true state of things, and in any question of this kind the dead load and wind must be considered independently, as in Fig. 1854, which is the frame diagram for a small roof. The force of the wind normal to the roof is first found, and then combined with the structural load to give an external force line. The stress diagram will then be found as shown in Fig. 1855. The funicular polygon in Fig. 1854, drawn parallel to the vectors in Fig. 1855, enables the load to be apportioned correctly between the two bearings. Taking a 9-in. wall 14 ft. high, as in Fig. 1856, the thrust from the roof, combined with the weight of the wall from truss to truss, shows a resultant passing 0·5 ft. outside the centre of base, and the calculation of effect will be as follows:—

$$\frac{\text{W}}{\text{A}} \pm \frac{\text{M}}{\text{Z}} = \frac{145}{10 \times \cdot 75} \pm \frac{145 \times \cdot 5}{\frac{1}{6}\,(10 \times \cdot 75^2)} = 19{\cdot}3 \pm$$

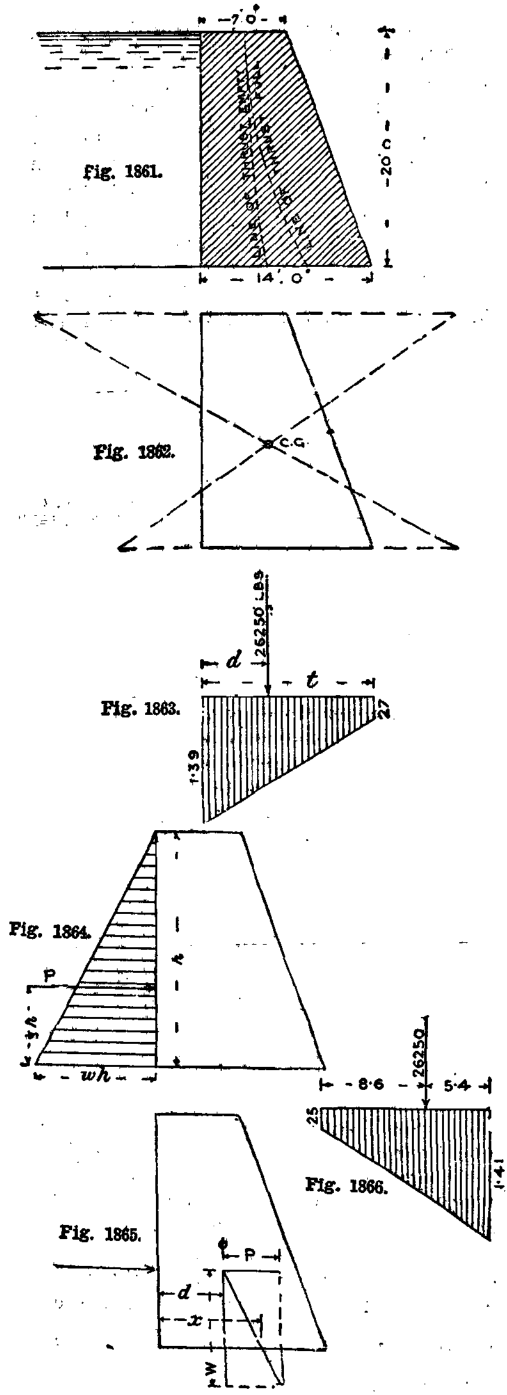
$77{\cdot}3$ = for the +, $96{\cdot}6 \div 20 = 4{\cdot}8\dot{3}$ tons per square foot compression, and for the −, $58{\cdot}0 \div 20 = 2{\cdot}9$ tons per square foot tension. The compressive stress might possibly be allowed, but the tensile stress is much too great, and the wall must be increased in thickness, or buttresses be added. Try a 14-in. wall, reducing the scale of stresses, as in Fig. 1857, then $\frac{W}{A} \pm \frac{M}{Z} = \frac{197\cdot5}{10 \times 1\cdot125} \pm \frac{197\cdot5 \times \cdot35}{\frac{1}{6}(10 \times 1\cdot125^2)} = 17\cdot\dot{5} \pm 32\cdot\dot{7} =$ for the +, $50\cdot3 \div 20 = 2\cdot5$ tons per square foot compression, and for the −, $15\cdot\dot{2} \div 20 = \cdot76$ tons per square foot tension, which are reasonable values, and about the maximum that ought to be permitted; therefore a 14-in. wall is necessary.



Figs. 1859 and 1860.—Comparison of Pressures on Either Side of Battering Wall.

Comparison of Pressures on Either Side of Battering Wall.

A long straight wall, built of, say, granite in cement mortar, having the vertical cross-section shown in Fig. 1858, is proposed to be built to retain water; it may be supposed to act as one block resting on a thin mortar bed at A B, which is supposed to be water-tight; the friction on A B is sufficient to prevent sliding on the bed, so that the tendency of the wall to fail is by overturning; the weight of the wall is 170 lb. per cubic foot; when the water stands at the level C D on the A C side of the wall, it is required to know what is the pressure in lb. per square foot on the bed at A, and what is the pressure in lb. per square foot on the bed at B. Again, suppose the wall to be built so that the side B D is next the water, it is to be ascertained what is now the pressure in lb. per square foot at A and at B. (See diagram Fig. 1859.) Area of section of wall $= \frac{(5 + 15)\,30}{2} = 300$ sq. ft. Weight of 1-ft. length of wall $= 300 \times 170 = 51000$ lb. The centre of gravity is found graphically as shown. The thrust of the water will be $\frac{1}{2}\,whl = \frac{1}{2} \times 62\cdot5 \times 27 \times 28\cdot4 = 23962\cdot5$ lb., acting at one-third of the height and perpendicular to the face. Completing the parallelogram of forces, the resultant scales as 63000 lb., of which the vertical component is 58650 lb., acting at 2·83 ft. from the outer edge. The maximum pressure on the edge will then be $\frac{2}{3} \times \frac{W}{d} = \frac{2}{3} \times \frac{58650}{2\cdot83} = 13800$ lb. per square foot, as shown graphically below the section. Refer now to diagram Fig. 1860 for water on the opposite side of the wall. Working in a similar manner to the above, the thrust of the water will be $\frac{1}{2} w h^2 = \frac{1}{2} \times 62{\cdot}5 \times 27^2 = 22781{\cdot}25$, the weight of wall will be as before, and the parallelogram will be as shown, where the vertical component of the resultant will be the weight of wall = 51000 lb. acting at 5·75 ft. from the outer edge, which, being more than one-third, will leave some pressure on both edges. These pressures will be $P = m \pm \frac{M}{Z}$, where m = mean pressure over area, M = bending moment = load (½ depth joint − dist. resultant from edge), Z = modulus of section $= \frac{b\,d^2}{6}$. Then $m = \frac{51000}{15} = 3400$, $M = 51000 \left(\frac{15}{2} - 5{\cdot}75\right) = 89250$ $Z = \frac{1 \times 15^2}{6} = 37{\cdot}5$, $P = 3400 \pm \frac{89250}{37{\cdot}5} = 3400 \pm 2380$, ∴ pressure at the outer edge = 3400 + 2380 = 5780 lb. per square foot, and pressure at the inner edge = 3400 − 2380 = 1020 lb. per square foot. When the resultant pressure is more than one-third of the thickness away from the outer edge, there is still a pressure on the inner edge. When it is exactly one-third away, the pressure on the inner edge is reduced to nothing, and at less than one-third there is either tension on the inner edge or a greatly increased pressure on the outer edge. If all contingencies have been allowed for, the resultant may be less than one-third from the outer edge, providing the maximum pressure does not exceed the safe working load on the material. Therefore, the only reason for keeping the resultant farther off the edge would be to increase the factor of safety.



Figs. 1861 to 1866.—Diagrams showing Pressure on Masonry Dam.

Pressure on Masonry Dam.

The following description explains how the maximum and minimum pressures at the foot of masonry dams are calculated (1) when the reservoir is empty, leaving no water pressure on the dam, and (2) when the reservoir is full; the resultant in the second case being taken as acting at an angle to the vertical. The line of thrust on the section of a masonry dam is usually plotted by finding the point of intersection of this line with horizontal beds at different levels from top to bottom, with the reservoir both full and empty. Assume a section (Fig. 1861) 20 ft. high, 7 ft. thick at top, 14 ft. thick at bottom, battering on the outside and vertical on the inside, water level with the top. Find the centre of gravity by adding the bottom width on each side of the top, and the top width on each side of the bottom, draw diagonals, and the intersection gives the centre of gravity as shown in Fig. 1862. Assume the stone to weigh 125 lb. per cub. ft., the area of the section will be $\frac{20\,(7 + 14)}{2} = 210$ sq. ft., and the weight $125 \times 210 = 26250$ lb., acting through the centre of gravity vertically downwards, at a distance of

$$\frac{20 \times 7 \times \frac{7}{2} + 20 \times \frac{7}{2} \times (\frac{7}{3} + 7)}{\frac{20\,(7 + 14)}{2}}$$

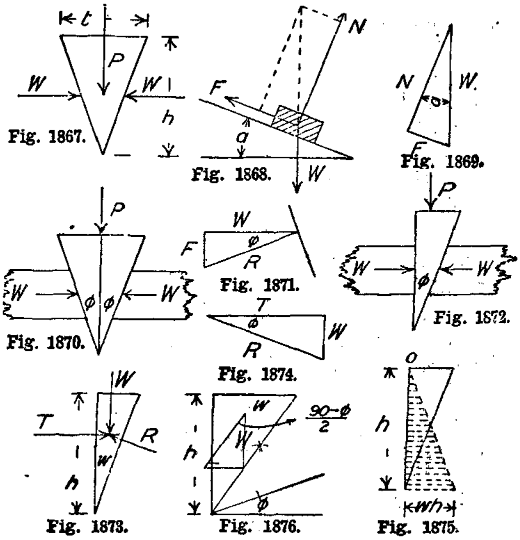
$= \frac{490 + 672}{210} = 5{\cdot}444$. Here, then, is a load of 26250 lb., acting at a distance of 5·444 ft. from the edge, on a base 14 ft. by 1 ft. The maximum pressure at the inner edge will be found by the formula

$$K = W\left(\frac{4\,t - 6\,d}{t^2}\right),$$

where K = maximum pressure per sq. ft. on the base in lb., W = resultant of superincumbent forces in lb., t = thickness of joint in feet, d = distance of resultant in feet from nearest edge of base. Then $K = 26250 \left(\frac{4 \times 14 - 6 \times 5{\cdot}444}{14^2}\right) = 3124{\cdot}95$ lb. $= 1{\cdot}39$ tons per sq. ft. maximum pressure on the inner edge when the reservoir is empty, the outer edge of course being much less, as given by the formula

$$k = \frac{W}{t}\left(\frac{6\,d}{t} - 2\right).$$

Then $k = \frac{26250}{14}\left(\frac{6 \times 5{\cdot}444}{14} - 2\right) = 624{\cdot}9$ lb. $= {\cdot}27$ tons per sq. ft., the intermediate pressure being as shown by the ordinates in Fig. 1863. When the reservoir is full the water presses with a force varying with the depth, being $62\frac{1}{2}$ lb. per sq. ft. per foot of depth ; or over the whole wall $P = \frac{1}{2}\,w\,h^2$, where P = total pressure on face of wall, w = weight of cub. ft. of water $= 62\frac{1}{2}$ lb., h = height of wall or head of water = 20 ft. Then $P = \frac{1}{2} \times 62\frac{1}{2} \times 20^2 = 12500$ lb. This acts at the centre of gravity of the ordinates of pressure as shown in Fig. 1864, at one-third the height from the base, and therefore with a leverage of $6\frac{2}{3}$ ft. This force must be combined with the weight of the wall acting through its centre of gravity as shown in Fig. 1865, the effect of which will be to shift the vertical load W to a distance $x = d + \frac{P\,h}{3\,W}$, because $W : P : \frac{1}{3}\,h : x - d$. Then $x = 5{\cdot}444 + \frac{1250 \times 20}{3 \times 26250} = 8{\cdot}6$ ft. Now must be considered the condition of Fig. 1866. where $K = 26250\left(\frac{4 \times 14 - 6 \times 54}{14^2}\right) = 3160{\cdot}7$ lb. $= 1{\cdot}41$ tons per sq. ft. maximum pressure on the outer edge, so that the section of wall at which the maximum pressures are about equal, under the varying conditions, on the inner and the outer face, has been rightly chosen.



Figs. 1867 to 1876.—Thrust on Retaining Walls shown Diagrammatically.

Thrust on Retaining Walls.

In order to lead up by easy stages to the action of a wedge of earth at the back of a retaining wall, it will be necessary to start with the mechanical principles of a simple wedge actuated by pressure. Let Fig. 1867 represent such a wedge, then, by the principle of velocity ratios, if the wedge is forced down the full depth h the horizontal movement caused will be measured by the thickness t, and, the pressures being inversely as the movements, we have P = force applied is to W = horizontal resistance overcome as t = thickness is to h = height, or $P\,h = W\,t$, whence $W = P \times \frac{h}{t}$; and if the wedge were without friction W would be equal to the horizontal thrust produced, all the movement being on one side only, or divided between the two sides. For the next step, consider the case of an inclined plane with the usual friction, as in Fig. 1868. The angle a is the "limiting angle of friction," or the angle at which the body would just commence to slide. Then the forces holding it in equilibrium are the weight W acting vertically, the friction F acting parallel to the plane, and the pressure N normal to the plane. By the parallelogram of forces the weight W may be resolved into the directions F and N, as shown by dotted lines. The amounts are also readily ascertained by the triangle of forces (Fig. 1869), the force lines being drawn parallel to their respective directions, and W giving the scale of the diagram, from which it will be seen that $F = N \tan a$. The expression $\tan a$ is called the "coefficient of friction," and is usually signified by the Greek letter mu (μ). Assuming a rough wedge to be composed of two inclined planes as in Fig. 1870, and that the insertion of the wedge produces a horizontal movement, then we have, by Fig. 1871, R = whole resistance consisting of W = useful resistance and F = friction. Then it is shown in Thornton's "Theoretical Mechanics (Solids)," p. 248, that $P = 2R (\sin\phi + \cos\phi \tan a)$, but $W = R \cos\phi$; therefore $R = \frac{W}{\cos\phi} = W \sec\phi$, then $P = 2W \sec\phi (\sin\phi + \cos\phi \tan a)$, but $\sec \times \sin = \tan$, and $\sec \times \cos = 1$; therefore $P = 2W (\tan\phi + \tan a)$, and not $P = 2W \tan(\phi + a)$ as in Garratt's "Principles of Mechanism," p. 156. Tracing the action through in detail, we have the pressure × distance moved = load × movement + length moved under friction × pressure normal to sides of wedge × coefficient of friction. Or $P h = W 2 h \tan\phi + 2 h \sec\phi \times W \cos\phi \times \tan a$, whence by cancelling $P = 2W (\tan\phi + \tan a)$ as before. Applying the same reasoning to a wedge with one side vertical as in Fig. 1872, we have $P h = W h \tan\phi + h \sec\phi \times W \cos\phi \times \tan a + h W \tan a$, whence $P = W (\tan\phi + 2 \tan a)$. For a similar wedge in equilibrium without friction, as in Fig. 1873, we have by triangle of forces (Fig. 1874) $T = W \tan(90 - \phi)$; and if the weight W be caused by the weight per unit mass w, we have $W = \frac{1}{2} w h^2 \tan\phi$, but $\tan(90 - \phi) = \cot\phi$, therefore $T = \frac{1}{2} w h^2 \tan\phi \cot\phi$; and since $\tan \times \cot = 1$, we obtain $T = \frac{1}{2} w h^2$. Suppose the wedge to be composed of water, which is without friction, it will exert a pressure against the vertical side due to its height only, irrespective of its thickness. This pressure = nil at the top, and $= w h$ at the bottom, as shown by ordinates to the dotted triangle (Fig. 1875); the total pressure will be $\frac{0 + w h}{2} \times h = \frac{1}{2} w h^2$, and the centre of pressure will be at $\frac{1}{3} h$ from the bottom; that is, the horizontal thrust will be $\frac{1}{2} w h^2$ acting at $\frac{1}{3}$ the height. Now, adopting the ordinary principle of calculating retaining walls, we have the natural slope of water $\phi = 0°$, the line of rupture bisecting the natural slope, and the vertical will be $\frac{90 - \phi}{2} = 45°$, and the wedge of water producing the thrust will have a weight of $\frac{1}{2} w h^2 \tan \frac{90 - \phi}{2}$, producing a thrust of $W \tan \frac{90 - \phi}{2} = \frac{1}{2} w h^2 \tan^2 \frac{90 - \phi}{2}$. But $\frac{90 - \phi}{2}$ being 45° $\tan 45° = 1$, and $\tan^2 45°$ also $= 1$, therefore $T = \frac{1}{2} w h^2$ as before. This establishes the common formula for retaining walls $T = \frac{1}{2} w h^2 \tan^2 \frac{90 - \phi}{2}$; and as the angle of natural slope increases, so the weight of wedge reduces, and the thrust upon retaining wall acting at one-third of the height becomes less, as in Fig. 1876. By Rankine's formula for the pressure of earth against a vertical wall, $\frac{\text{horizontal pressure}}{\text{vertical pressure}} = \frac{1 - \sin\phi}{1 + \sin\phi}$ ϕ being the natural slope or angle of repose. From this it results that the horizontal pressure = vertical pressure $\times \frac{1 - \sin\phi}{1 - \sin\phi}$. At the top of wall and surface of unsupported earth, both pressures are zero; at the bottom the vertical pressure $= w h$; therefore the horizontal pressure at the bottom $= w h \frac{1 - \sin\phi}{1 + \sin\phi}$. The mean horizontal pressure will then be $\frac{1}{2} w h \frac{1 - \sin\phi}{1 + \sin\phi}$, and the *total* horizontal pressure or thrust $T = \frac{1}{2} w h^2 \frac{1 - \sin}{1 + \sin\phi}$ acting at $\frac{1}{3} h$.

The ratio $\frac{1 - \sin \phi}{1 + \sin \phi} = \tan^2 \frac{90 - \phi}{2}$, but is in a simpler form, as it does not involve the squaring of a decimal quantity containing several figures.

Stability of Retaining Wall.

To find the resultant of the pressure and resistance, draw the outline of the wall as shown in Fig. 1877, assuming the thickness at the base to be 7 ft. Assume the natural slope at 45°, and bisect the angle formed by the slope with back of wall in order to give the line of rupture. Find the weight (or the comparative weight) of the wedge of earth acting through its centre of gravity, also find the weight (or the comparative weight) of the wall acting through its centre of gravity, and draw the parallelogram of thrust and resistance in order to give the resultant. If the specific gravity of the earth and the wall be taken at $1\frac{2}{3}$ and $2\frac{1}{4}$ respectively, the resultant will cut 2 ft. within the outer edge of the base, which is a reasonable position.

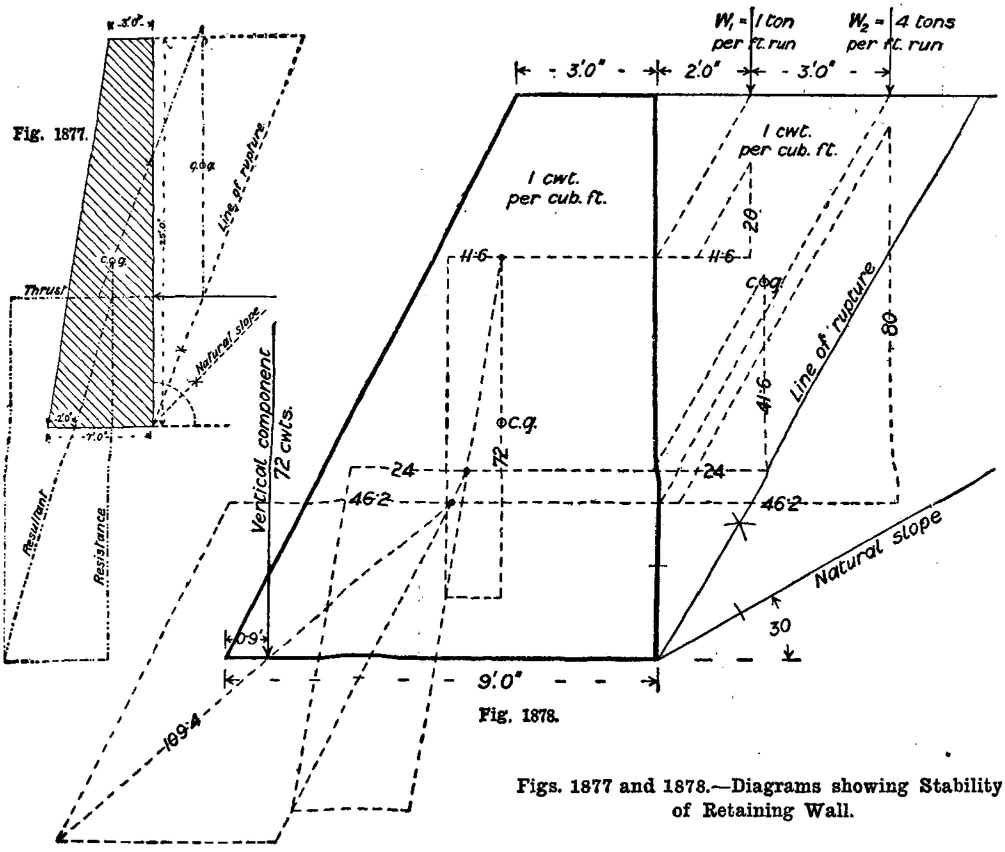


Fig. 1877.
3'0"
Line of rupture
Natural slope
25'0"
c.g.
Thrust
2'0"
7'0"
Resultant
Resistance
Vertical component
72 cwts.
W₁ = 1 ton per ft. run
W₂ = 4 tons per ft. run
3'0"
2'0"
3'0"
1 cwt. per cub. ft.
1 cwt. per cub. ft.
28
11·6
11·6
c.g.
c.g.
80
41·6
Line of rupture
24
24
72
46·2
46·2
Natural slope
30
9'0"
·9
109·4
Fig. 1878.

Figs. 1877 and 1878.—Diagrams showing Stability of Retaining Wall.

Retaining Walls with Loads on Surface at Back.

To find a suitable section for retaining walls where the weight of superincumbent buildings has to be taken into account, the following method is put forward as a suggestion that appears reasonable: Assume the height of wall to be 12 ft., and the natural slope of earth 30°, the weight of brickwork and earth each to be 1 cwt. per cubic foot, and two loads to be carried, W_1 of 1 ton per foot run at 2 ft. from back of wall, and W_2 of 4 tons per foot run at 5 ft. from back of wall. Then the distance from back of wall to outcrop of line of rupture will be $h \tan \frac{90 - \phi}{2} = 12 \times {\cdot}5774$

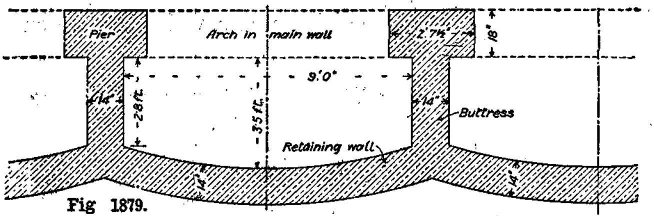

Pier
Arch in main wall
2'7½"
18"
9'0"
14"
2·8 ft.
3·5 ft.
14"
Buttress
Retaining wall
14"
14"

Fig 1879.

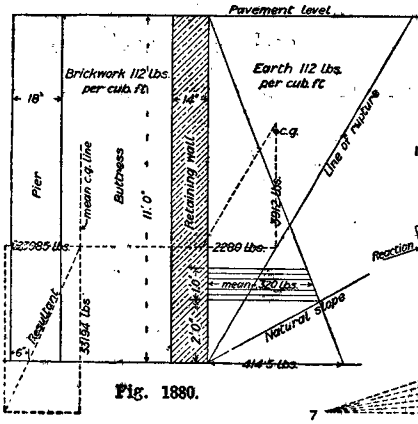

Pavement level
Brickwork 112 lbs. per cub. ft.
18"
14"
Earth 112 lbs. per cub. ft.
Line of rupture
c.g.
Pier
mean c.g. line
Buttress
11'0"
Retaining wall
5912 lbs.
323985 lbs.
2280 lbs.
mean 320 lbs.
1'0"
2'0"
Natural slope
Resultant
33194 lbs
6"
414·5 lbs.

Fig. 1880.

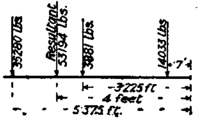

35280 lbs
Resultant 53194 lbs.
3881 lbs.
14033 lbs
7"
3·225 ft
4 feet
5·375 ft.

Fig. 1883.

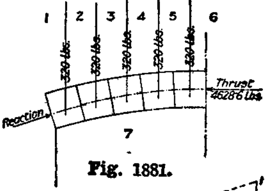

1
2
3
4
5
6
320 lbs.
320 lbs.
320 lbs.
320 lbs.
320 lbs.
320 lbs.
Reaction
Thrust
4628·6 lbs.
7

Fig. 1881.

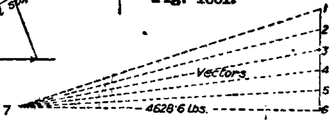

1
2
3
4
5
6
Vectors
4628·6 lbs.
7

Fig. 1882.

Figs. 1879 to 1883.—Diagrams showing Strength of Arched and Buttressed Wall.

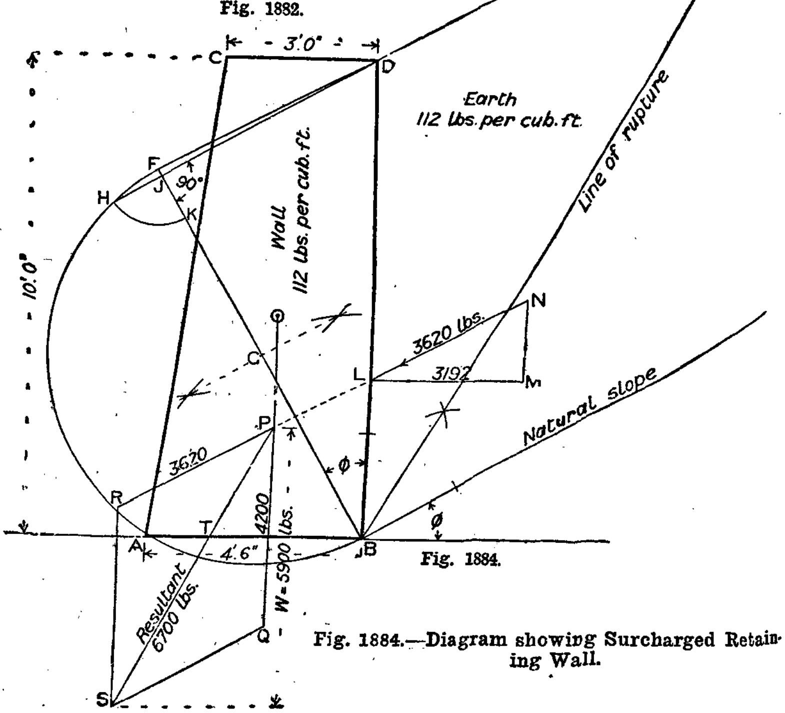

E
Slope of surcharge
C
3'0"
D
Earth 112 lbs. per cub. ft.
F
H
J
90°
K
Wall 112 lbs. per cub. ft.
Line of rupture
10'0"
O
N
3620 lbs.
C
L
3192
M
P
3620
φ
Natural slope
R
T
4200
lbs.
φ
A
4'6"
B
Resultant 6700 lbs.
W=5900
Q
S

Fig. 1884.

Fig. 1884.—Diagram showing Surcharged Retaining Wall.

= 6·9288 ft., the weight in cwt. of wedge of earth will be $\frac{1}{2} w h^2 \tan \frac{90 - \phi}{2} = \frac{1}{2} \times 1 \times 12^2 \times {\cdot}5774 = 41{\cdot}6$ cwt., and the thrust $\frac{1}{2} w h^2 \tan^2 \frac{90 - \phi}{2} = \frac{1}{2} \times 1 \times 12^2 \times {\cdot}5774^2 = 24$ cwt., acting at a height of $\frac{1}{3} h = \frac{12}{3} = 4$ ft. Assuming that the external loads produce a thrust at a point where a line from their point of application parallel to the line of rupture will cut the back of the wall as shown, W_1 will produce a thrust at a depth of $d \tan (90 - \phi) = 2 \times 1{\cdot}73 = 3{\cdot}46$ ft. from top, amounting to $\text{W}_1 \tan \frac{90 - \phi}{2} = 20 \times {\cdot}5774 = 11{\cdot}548$, say 11·6 cwt., and W_2 will produce a thrust at a depth of $\text{D} \tan (90 - \phi) = 5 \times 1{\cdot}73 = 8{\cdot}65$ ft. from top, amounting to $\text{W}_2 \tan \frac{90 - \phi}{2} = 80 \times {\cdot}5774 = 46{\cdot}192$, say 46·2 cwt., d and D being distances from back of wall. Now assume a wall of probable section, say 3 ft. thick at top and 9 ft. at bottom (see Fig. 1878), the weight will be $\frac{1}{2} (t + b) h w = \frac{3 + 9}{2} \times 12 \times 1 = 72$ cwt., and the distance of the centre of gravity line from the back of the wall will be $\frac{th \times \frac{1}{2}t + \frac{1}{2}(b - t) h \left(t + \frac{b - t}{3}\right)}{\frac{1}{2}(t + b) h} = \frac{54 + 180}{72} = 3{\cdot}25$ ft. The remainder can be worked by parallelogram of forces as shown, finding first the resultant of the top thrust and weight of wall, then combining this resultant with the next thrust, and the second resultant with the lower thrust for the final resultant. The vertical component of this will be the load on the base joint acting where the final resultant cuts. Or it can be calculated by moments round the bottom corner of the back of the wall, thus $\frac{\text{T}_1 \text{H}_1 + \text{T} \frac{1}{3} h + \text{T}_2, \text{H}_2 + \text{W}_3 \text{D}_3}{\text{W}_3} = 11{\cdot}6 \times (12 - 3{\cdot}46) + 24 \times 4 + 46{\cdot}2 \times (12 - 8{\cdot}65) + 72 \times 3{\cdot}25 \div 72 = \frac{99{\cdot}064 + 96 + 154{\cdot}77 + 234}{72} = \frac{538{\cdot}834}{72} = 8{\cdot}11$ ft. distance of resultant from the back of the wall, or $9 - 8{\cdot}11 = 0{\cdot}9$ from the toe. Then $\frac{2}{3} \frac{\text{W}}{\text{D}} = \frac{2 \times 72}{3 \times 0{\cdot}9} = \frac{144}{2{\cdot}7} = 53{\cdot}3$ cwt. = 2·66 tons per square foot maximum pressure at the outer edge. The usual safe load on brickwork in mortar is 3 tons per square foot, so that the wall is just right. Had the thickness assumed not been suitable, half the work would have had to be done over again until a suitable section was found. The great thickness shown in Fig. 1878 is a consequence of the heavy loads assumed to pass behind the wall, which can be modified to suit any given circumstances. If a building occurs, the centre of the foundation at the under side will give the point for the line parallel to line of rupture. In the case of a timber dock wall, with crane or locomotive at back, the same general procedure may be adopted, the pile being treated as a cantilever subject to loads where the thrusts occur. The line of rupture or cleavage of the earth is in all cases half the angle between the natural slope and a vertical line, and not the bisection of the natural slope with the back of the wall. Any load that lies on the earth at the back outside the line of rupture may be ignored.

Strength of Arched and Buttressed Wall.

The arched and buttressed wall is commonly used in the construction of the vaults or cellars of city buildings. Fig. 1879 shows the general plan, and the curved portion may first be dealt with as a single arch. The first step is to find the maximum load per foot run the arch will have to support, but some preliminary work is necessary. In the vertical section of the wall (Fig. 1880), the nature of the earth being given as "ordinary good soil," assume a natural slope of, say, 30°, bisect the angle between the back of the wall and the natural slope, giving the line of rupture. Then, taking the weight of earth as 112 lb. per cubic foot, the wedge will weigh $\frac{11}{2} \times 6{\cdot}35 \times 112$ = say 3912 lb. per foot run of length. This being worked in the usual manner gives a thrust at back of wall of 2,280 lb. acting at one-third the height. The pressure per square foot at the base of the wall will be $\frac{\text{T}}{\frac{1}{2}h} = \frac{2280}{\frac{1}{2} \times 11} = 414{\cdot}5$ lb. Set this out at right angles to the base of the wall, and join by a straight line to the top of the wall; then the pressure at any point will be the ordinate of this triangle at the height required. The next step is to find the pressure per square foot at the back of the wall. It would obviously be unfair to take the bottom square foot, as although the pressure is greatest the resistance is also great; but we may take a depth of 1 ft. at, say, 2 ft. up the wall. This will give a mean pressure of 320 lb. per square foot. Now draw the elevation of half the arch as in Fig. 1881, and divide it up into voussoirs 1 ft. wide, as shown. Find the centre of gravity of each voussoir, and draw a short vertical line through each centre of gravity. Having obtained the load per square foot on the arch, the next thing is to find the horizontal thrust at the crown. This will be $\frac{w\,l^2}{8\,r}$, where w = load per foot run in pounds, l = span in feet, and r = rise of arch in feet, $\frac{320 \times 9^2}{8 \times \cdot 7} = 4628{\cdot}6$ lb. at the crown. Now number the spaces 1 to 7 as Fig. 1881, and draw the load line 1 to 6 as shown in Fig. 1882. From point 6 set out a horizontal line equal to the thrust at the crown. This will fix point 7, from which vectors may be drawn to each of the points in the load line. Parallel to these vectors draw line of thrust as shown by the stroke-and-dot line in Fig. 1881; and as this line of thrust keeps well within the middle third, we may assume that the wall will be safe as regards its treatment as an arch, the intensity of the thrust being $\frac{4628{\cdot}6}{1{\cdot}125 \times 1 \times 2240} = 1{\cdot}837$ tons per square foot. The next thing is to see whether the pier and buttress will be sufficiently strong to resist the thrust of the earth. As already found, the total thrust per foot run is 2280 lb., acting at one-third the height; and as the piers are 10-ft. 1½-inch centres, the total thrust will be 2280 × 10·125 = 23085 lb. No particulars are given as to the load on the pier from the superstructure; but assuming a load of 4 tons per square foot at the base, including the weight of the pier itself, we have 2·625 × 1·5 × 4 × 2240 = 35280 lb. acting down the centre of the pier at a distance of 5·375 feet from back of wall A. The weight of the buttress will be 11 × 1·125 × 2·8 × 112 lb. = 3881 lb. acting at a distance of 3·225 ft. from A, and the weight of the retaining wall will be 11 × 1·125 × 10·125 × 112 lb. = 14033 lb. acting at, say, 0·7 ft. from A. The mean centre of gravity line will be found to be about 4 ft. from A, and the total vertical load 53,194 lb. as in Fig. 1883. Produce the thrust line in Fig. 1880 until it cuts the mean centre of gravity line, and from the intersection set out 23,085 lb. horizontally and 53,194 lb. vertically. Complete the parallelogram, and draw the resultant, which will be found to cut the base at a distance of 6 in. from the outside of the pier. Then by the formula $\frac{4}{3}\,\frac{\text{W}}{\text{D}}$ for maximum pressure, we have $\frac{4}{3} \times \frac{53194}{\cdot 5} =$ 70925 lb. = 31·6 tons per square foot, which would only allow a very small factor of safety, say less than 2, even with the brickwork in cement and best workmanship. Probably the actual load on the pier is less than is assumed above. This will throw the mean centre of gravity farther back, and reduce the vertical component of the parallelogram; but the particulars given above will enable any required correction to be made for actual loading.

Surcharged Retaining Wall.

Assuming the case shown in Fig. 1884, if the wall had been without surcharge, the thrust would have been $\text{T} = \frac{1}{2}\,w\,h^2 \tan^2\left(\frac{90 - \phi}{2}\right) = \frac{1}{2} \times 112 \times 10^2 \times {\cdot}5774^2 = 1866{\cdot}6$ lb. Taking the effect of the surcharge as merely so much extra weight in the wedge of earth, and working in the usual way for a horizontal top, the thrust would be 2,740 lb. Taking the thrust by Rankine's graphic method, as shown in the accompanying section, it will amount to 3,192 lb. horizontally, or 3,620 lb. acting at the same angle as the slope of the surcharge. This diagram may be considered to give the correct thrust, and is constructed as follows: Let A B C D be the section of the proposed wall, D E the line of surcharge, the natural slope and line of rupture being drawn as usual. Then set up B F making angle ϕ with back of wall, produce E D to F; bisect B F in G, and from G as centre draw the semicircle B F. From D drop a perpendicular on to B F, cutting it in J, and continue D J through to H. From centre F, with radius F H, cut B F in K. Then the horizontal thrust will be found by the formula $\text{T} = \frac{1}{2}\,w\,(\text{B K})^2 = \frac{1}{2} \times 112 \times 7{\cdot}55^2 = 3192$ lb. acting at one-third of the height as shown at M L. Draw L N parallel with the line of surcharge, and cut it off by the vertical M N, then N L gives the actual thrust against the wall. Drop a line through the centre of gravity of the wall, and produce N L to intersect it in P; then P R =

thrust, and P Q = weight of wall, the resultant P S cuts the base at T, 1·35 ft. from the outer edge, the vertical component being 5,900 lb. Then the maximum stress on the outer edge of base, the resultant being outside the middle third, will be $\frac{2}{3}\,\frac{W}{d} = \frac{2}{3} \times \frac{5900}{1\cdot35} \div 2240 = 1\cdot3$ tons per square foot, no tension being allowed on the inner edge. The maximum safe load on brickwork being from 3 to 10 tons per square foot, according to quality, there is an ample margin of safety, but if the wall is reduced in thickness the maximum stress will rapidly increase, so that it may be left as given.

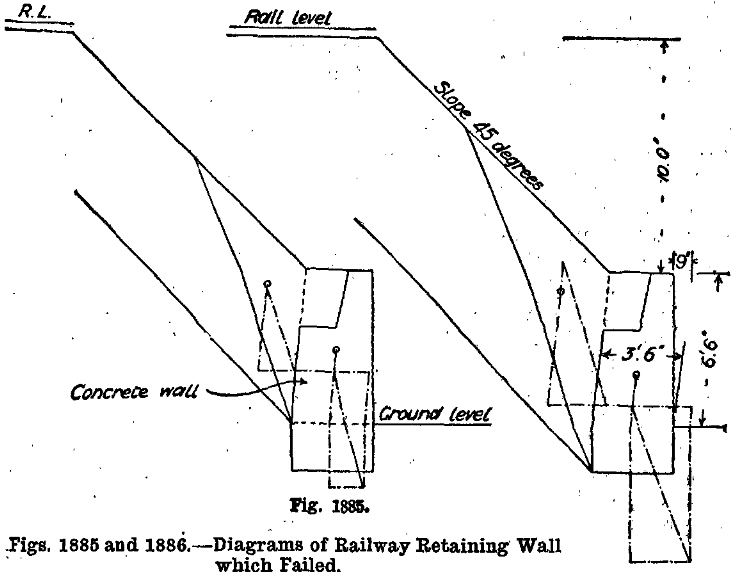


Fig. 1885.

Fig. 1886.

Figs. 1885 and 1886.—Diagrams of Railway Retaining Wall which Failed.

Railway Retaining Walls.

The recent failure of a surcharged retaining wall affords a good object lesson of the difficulties to be encountered in using such walls. Fig. 1885 shows a section of the original wall and embankment. A length of 500 yd. was so constructed, and one-fifth of this has now failed by the top being pushed forward, so that the face is nearly vertical. It is said that this result is due to water getting behind and increasing the pressure beyond the safe limit, but this contingency is generally provided for by weep-holes. A dwarf wall in this position is usually called a breast wall, and is often looked upon as only having to support a mass of earth equivalent to that displaced by the wall from the toe of the embankment. It is, however, in reality a surcharged retaining wall, and subject to the same laws of stability. If it were leaned back against the embankment at the same slope there would, of course, be no thrust against it; but there would be no advantage, as the object is to widen the space at the foot by curtailing the slope of the embankment. Upon testing the stability by the usual graphic method, it is found that the line of thrust comes within the middle third of base at ground level, as shown in Fig. 1885, so that by ordinary rules it should have been perfectly safe. The fault may be that the wall has no front toe or footing, and this is tested in Fig 1886 by taking the whole wall as receiving a surcharged thrust at the back. The counter thrust on the lower part of the face was allowed for, but has been omitted from the figure, being too small to show, as it only amounted to 25 lb. or 30 lb. It will be seen that the final resultant cuts the base at one-fourth from the front edge, and that the pressure on the earth at what should be the toe of the wall reaches 1·11 tons per square foot. This is assuming the natural slope to be the same as the embankment slope, 45°, and the earth and concrete to be the same weight. If weep-holes were provided, the design appears to have been suitable; but the failure occurred

ARCHITECTURE: NOTES AND EXAMPLES.

Introduction.

In this section it is proposed to deal with the artistic part of Building Construction, and the notes and examples given are classified so as to extend—as far as possible chronologically—from the earliest times to the present day. In many cases the notes take the form of model solutions to examination questions.

Short Sketch of Greek Architecture.

Greek architecture may be divided into three periods: its archaic state, its perfected condition, and its decadence. The first period owes its development to the earliest settlers in Greece, who were in course of time displaced by other migrations from the East. The first period, the archaic, dates from the twelfth century B.C. The second period, from 450 B.C. to 300 B.C., is the great temple-building period in Greece and also some of her colonies, as Sicily and the Archipelago. During the last-mentioned period some of the finest works of Greek art, sculpture, and painting were executed. After this period there was a long lapse, and then Greece came under the domination of the Romans. The Romans built other temples in the conquered lands, but they transported her chief treasures to Rome. The period of decadence then seemed to begin. The earliest example of a temple of the archaic order is the Doric Temple of Corinth, of which only a few columns remain. It is supposed that the Egyptian temple formed the model from which all other structures borrowed their leading features. The Egyptian temples were enclosed with lofty walls, but the Greek temple was to be seen from all sides, and appeared isolated. To give the front of the temple a better appearance, or to afford protection to the paintings, by prolonging the side walls and providing two columns to carry the entablature, a porch would be formed. This gave the temple a bold appearance, and many temples were found to be built in this way. Sometimes four to six columns would be placed on the front, while occasionally a temple with columns all round it would be found. The temple was placed on a base consisting of three steps, varying, according to the size of the temple, from 6 in. to 18 in. or more. The plans of these temples were of the simplest and most elementary character. The roof was constructed of timber, with a covering of Parian marble, in imitation of tiles. The columns taper upwards with a slight curve, instead of being straight.

Four Periods of Greek Architecture.

First Period.—Cyclopean, Early Doric, and Ionic, 1184 B.C. to time of Solon. Examples: Ruins in Greece, Italy, Sardinia; treasure house of Atreus at Mycenæ; temple at Corinth; temples of Here and Olympia at Samos.

Second Period.—Extends from 600 B.C. to the time of Pericles, who died B.C. 429. Examples: Temples of Olympic Zeus at Athens and Apollos at Delphi, both Doric; Temple of Diana at Ephesus, Ionic; temples at Syracuse, Selinus, Agrigentum, and Egesta, all in Sicily; Temple at Pæstum, 500 B.C.; Temple of Minerva at Egina, 479 B.C., all Dorian, no Ionic building of this period having existed till the present day.

Third Period.—Extends from the time of Pericles to that of Alexander the Great (died B.C. 323). Examples: Temple of Pallas Athene, or the Parthenon, on the Acropolis, and the gate called Propylæa, both at Athens; temples at Eleusis, Rhampos, Senion, and Thorikos (mostly Doric); the triple temple of Erechtheum (in the Ionic style); Temple of Apollo at Bassæ in Arcadia (Doric, Ionic, and Corinthian styles all introduced); Temple of Minerva Alea at Tagea (Doric, Ionic, and Corinthian), of which Scopas the Athenian was architect; choragic monument of Lysicrates at Athens.

Fourth Period.—Extends from the period of

Alexander the Great to 150 A.D. Examples: Temple of Apollo Didymæus at Miletus, Temple of Jupiter Stator at Rome, theatre at Jassus.

Cyclopean Architecture.

The earliest examples of Grecian architecture are the remains of the circular walls round towns and palaces known as Cyclopean. They consist of huge masses of stone put together without mortar, and may be divided into three varieties: (1) Enormous unhewn boulders in their natural shape laid one on another, and the interstices filled up with smaller stones. (2) Gigantic polygonal blocks

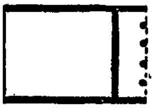


Fig. 1887.

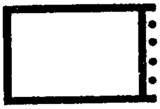


Fig. 1888.

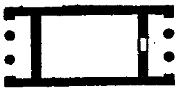


Fig. 1889.

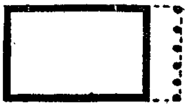


Fig. 1890.

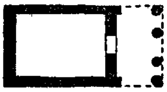


Fig. 1891.

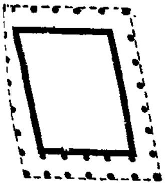


Fig. 1892.

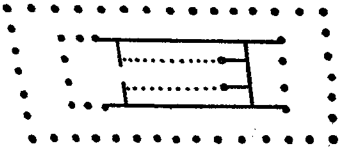
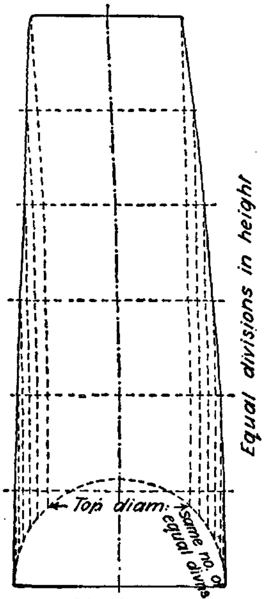
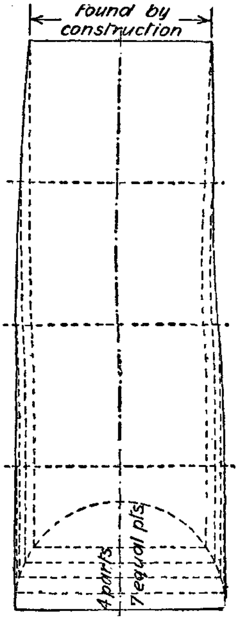
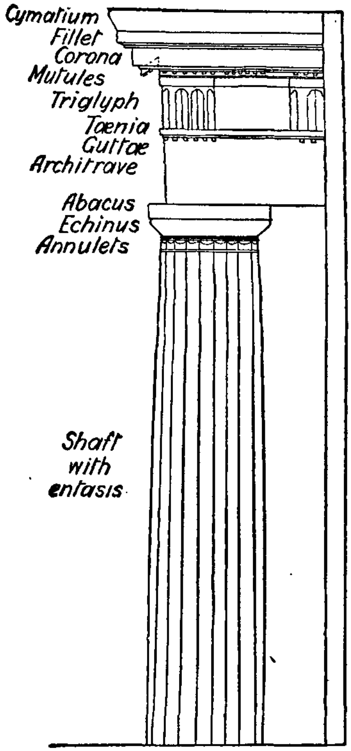
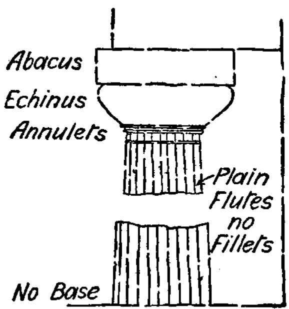


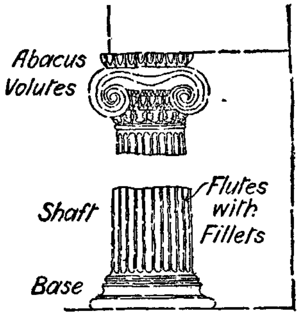
Fig. 1893.


Equal divisions in height
Top diam.
Same no. of equal divns
Fig. 1894.


found by construction
4 parts
7 equal pts
Fig. 1895.


Cymatium
Fillet
Corona
Mutules
Triglyph
Taenia
Guttae
Architrave
Abacus
Echinus
Annulets
Shaft with entasis
Fig. 1896.


Abacus
Echinus
Annulets
Plain Flutes no Fillets
No Base
Fig. 1897.


Abacus
Volutes
Shaft
Flutes with Fillets
Base
Fig. 1898.

Figs. 1887 to 1889.—Plans of Temple "in Antis." Figs. 1890 and 1891.—Prostyle.—Figs. 1892 and 1893. —Pseudo-dipteral. Figs. 1894 and 1895.—Entasis. Figs. 1896 to 1898.—Distinguishing Features of Columns.

of stone, the corners of which fit accurately into one another. (3) Large regular blocks of equal height, approximately squared, and forming what we should now call heavy ashlar work.

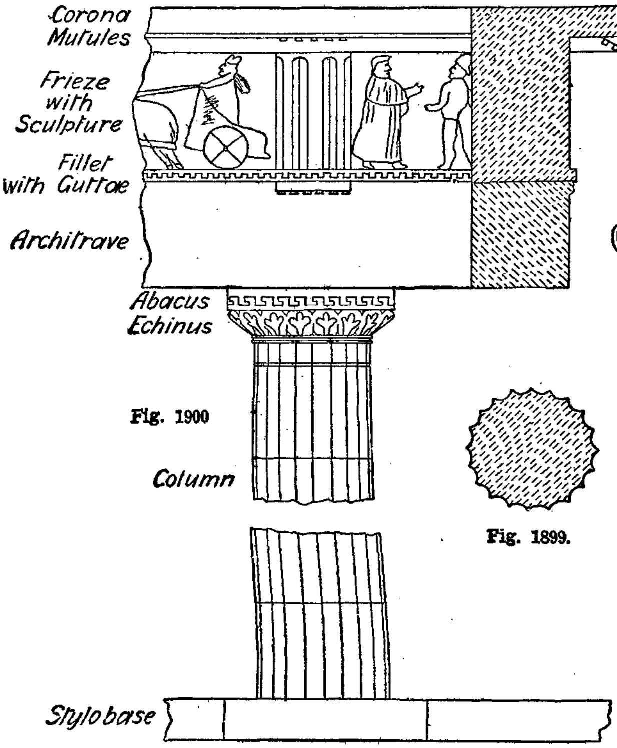

Corona
Mutules
Frieze with Sculpture
Fillet with Guttæ
Architrave
Abacus
Echinus
Column
Stylobase

Fig. 1900

Fig. 1899.

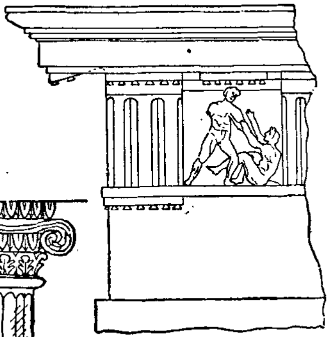


Fig. 1903.

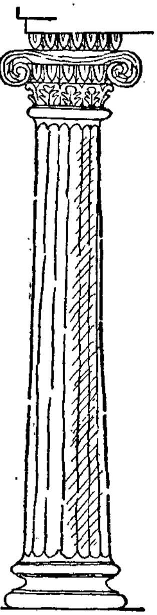


Fig. 1902.

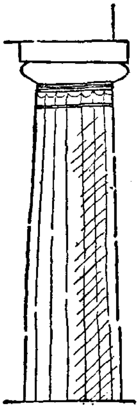


Fig. 1901.

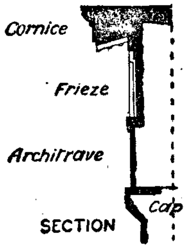

Cornice
Frieze
Architrave
SECTION
Cap

Fig. 1904.

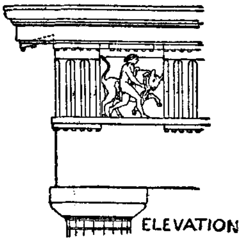


Fig. 1905.

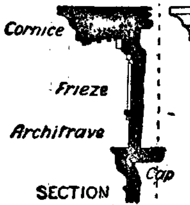

Cornice
Frieze
Architrave
SECTION
Cap

Fig. 1906.

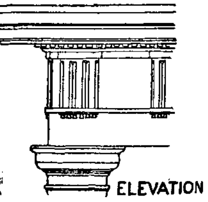


Fig. 1907.

Figs. 1899 and 1900.—Doric Column with Part of Architrave and Frieze. Fig. 1901.—Doric Column. Fig. 1902.—Ionic Column. Figs. 1903 to 1905.—Greek Doric Entablature. Figs. 1906 and 1907.—Roman Doric Entablature.

The First Grecian Doric Temple.

It is still a matter of conjecture in what city the first Grecian Doric temple was erected; but Vitruvius states that Dorus, the son of Helen, reigned over Achaia and Peloponnesus, and that he built a temple of the Doric order on a spot sacred to Juno in the ancient city of Argos, 1,200 years B.C. The most ancient existing specimen of a temple of the Doric order is that at Corinth, known as the Temple of Athene, of which there are only seven columns left standing. It is a simple, plain, bold design of massive proportions. The date of building is about 650 B.C. The Ionic temple of Fortuna Virilis, now existing as the

Church of Santa Maria Egiziaca, is one of the few Ionic buildings in Rome. It is tetrastyle, with half-columns all round it, or "pseudoperipteral," as Vitruvius termed it; date about 140 B.C. The Temple of Concord in Rome is another specimen of Ionic design.

The Planning of Temples.—The arrangement of the columns gives the key to the planning. "In antis" is the term applied to a temple where the side walls are continued beyond the front wall and the columns placed between as Fig. 1887, or where the front is terminated by antæ or pilasters and the columns are placed between, as in Figs. 1888 and 1889. "Prostyle" is the next step in the arrangement of the columns, and consists in advancing the pediment in front of the main building, the pediment and entablature being carried on a row of columns open at the ends, as in Fig. 1890; or when the columns are advanced in front of the antæ, as Fig. 1891. "Pseudo-dipteral" is as if dipteral or with double row of columns on each flank, whereas there is only a single row of columns on each flank, giving more space within the colonnades than the dipteral. There is a double row of columns in front, and the others are placed the same distance from the cella as the outer row, and therefore give a pseudo or false appearance, as Figs. 1892 and 1893. "Hypæthral" is applied to those temples having the cella or naos partially or wholly open to the sky.

Entasis of a Column.

The entasis of a column is the swelling of the diameter beyond the outline which would be formed by a straight taper. "Some authorities make it consist in preserving the cylinder of a column perfect for one-quarter or one-third the height of the shaft from below, diminishing thence in a right line to the top; while others, following Vitruvius, make the column increase in bulk in a curved line from the base to three-sevenths of its height, and then diminish in the same manner for the remaining four-sevenths, thus making the greater diameter near the middle." According to another account, "the shaft diminishes in a slightly curved line from the base, or inferior diameter, upwards to the hypotrachelium, leaving it at that place from two-thirds to four-fifths of the diameter at base." Tredgold's outline of the entasis was obtained by setting off the height and the top and bottom diameter of the column, describing a semicircle on the bottom and dividing the portion on each side that extended beyond the top diameter into any number of equal parts, then dividing the height into the same number of equal parts, and projecting vertical lines from the divisions on the semicircle to cut those in the height at points in the curve, as in Fig. 1894. Another modification was to divide the centre line of the semicircle into seven equal parts, making the top diameter the same dimension as the third line down, projecting vertical lines from the other divisions as before (see Fig. 1895). The swelling of the column makes it look as if actually supporting the load on top and swelling under the pressure as wrought iron does in compression. It also prevents the sides of the shaft from looking hollow. The taper increases the effect of height by the appearance of diminished size due to distance, and a larger diameter at the bottom is scientifically accurate to compensate for the weight of the column itself.

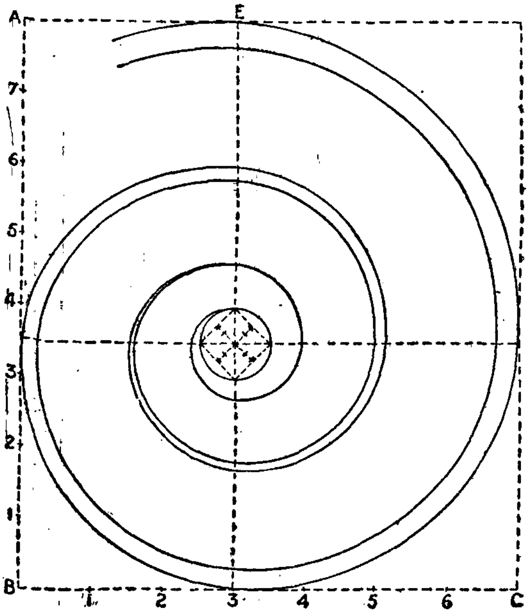


Fig. 1908.—Setting out Ionic Volute.

Grecian Doric Columns and Entablature.

Entablature in Greek and Roman architecture is the name given to the whole of the parts of an order above the column. It is sub-

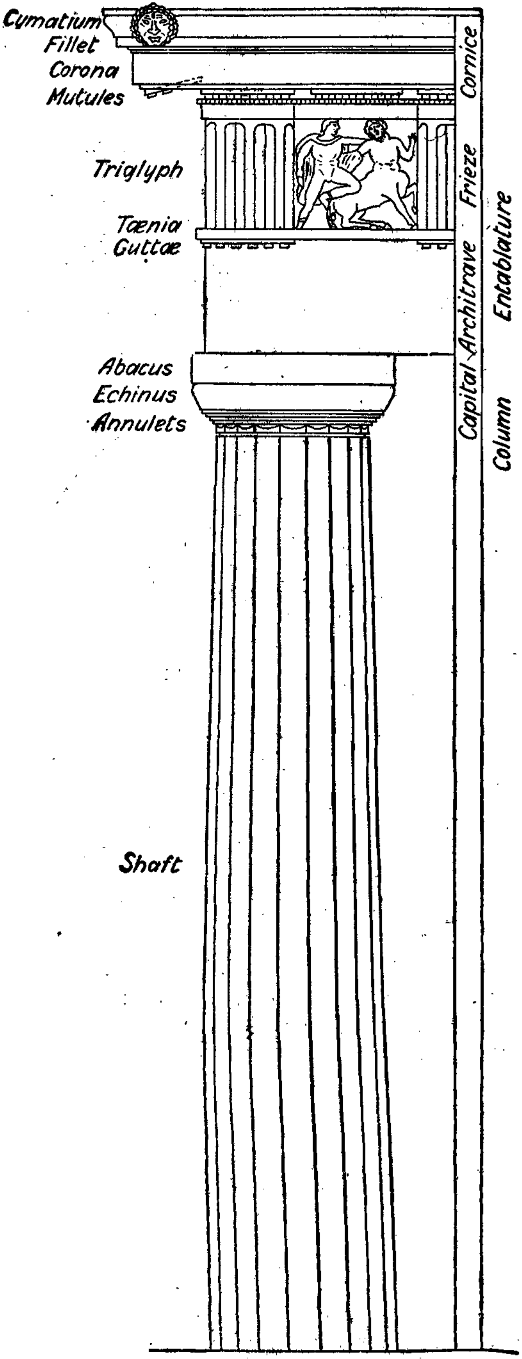

Cymatium
Fillet
Corona
Mutules
Triglyph
Tænia
Guttæ
Abacus
Echinus
Annulets
Shaft
Cornice
Frieze
Architrave
Capital
Entablature
Column

Fig. 1909.—Doric Order.

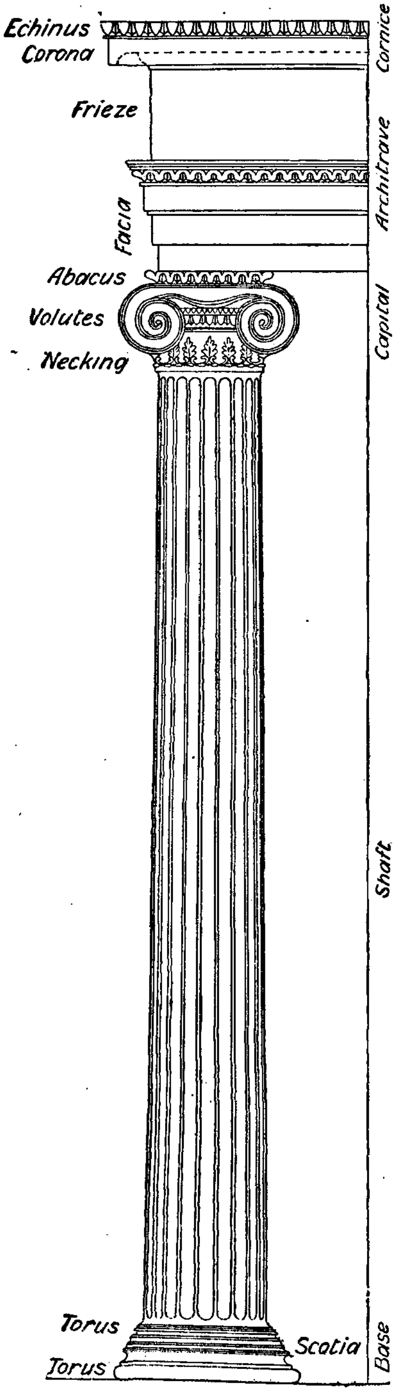

Echinus
Corona
Frieze
Facia
Abacus
Volutes
Necking
Torus
Scotia
Torus
Cornice
Architrave
Capital
Shaft
Base

Fig. 1910.—Ionic Order.

divided into three parts, the architrave resting immediately on the column, the frieze or flat surface separating the architrave from the cornice which forms the outer group. According to Gwilt, the proportions of the columns and entablature of the Greek Doric order in modules and minutes (a module being the semidiameter of column at base, and a minute the thirtieth of a module) are as follows:—

Height of column = 16 modules = 8 diameters.

Height of entablature { architrave, 30 min. / frieze, 45 min. / cornice, 45 min. } = 4 modules

According to other authorities, the column is from 8 to 12 modules high, the capital and necking occupying two of these; and the entablature 3½ to 4 modules high, of which architrave occupies two-fifths, frieze two-fifths, and cornice one-fifth (Figs. 1896 to 1898). The diameter of a Doric column being taken as 4 ft., Figs. 1899 and 1900 represent section and elevation, the latter showing Greek treatment of architrave and frieze. The distinctive features of Doric columns are:—Plain square abacus, saucer capital or quirked ovolo, neckings or grooves at top of shaft, twenty flat flutes worked to sharp arris. No base. Column 5 to 6½ diameters long (see Fig. 1901). The distinctive features of Ionic columns are:—Small ornamented abacus. Capital with large volutes at sides or angles, with egg-and-dart moulding, torus moulding separating shaft from capital, twenty-four semicircular flutes with fillet between; base consisting of two torus mouldings separated by scotia and two fillets. Column 7½ to 9 diameters long (see Fig. 1902). Special features of the Doric entablature are:—(1) Architrave quite plain, simple flat surface. (2) Frieze ornamented by triglyph and square sculptured metopes between, with mutules above both, and guttæ below triglyphs. (3) Projecting cornice of shallow depth, with mutules on under side. For Greek Doric entablature, see Figs. 1903 to 1905, the latter being from the Parthenon. For Roman Doric, from the Theatre of Marcellus, see Figs. 1906 and 1907. Figs. 1901 and 1902 are only rough memory sketches.

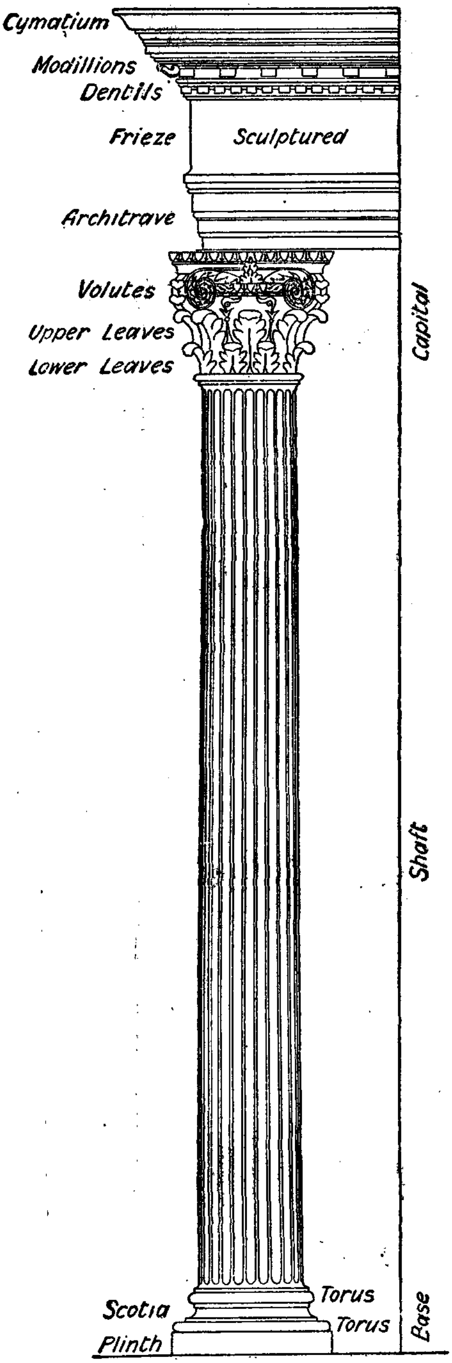


Fig. 1911.—Composite Order.

Greek Temples Outside Greece.

At Agrigentum, the Doric temple of Juno Lucina, the Doric temple of Concord, the temple of Peace, the temple of Jupiter: at

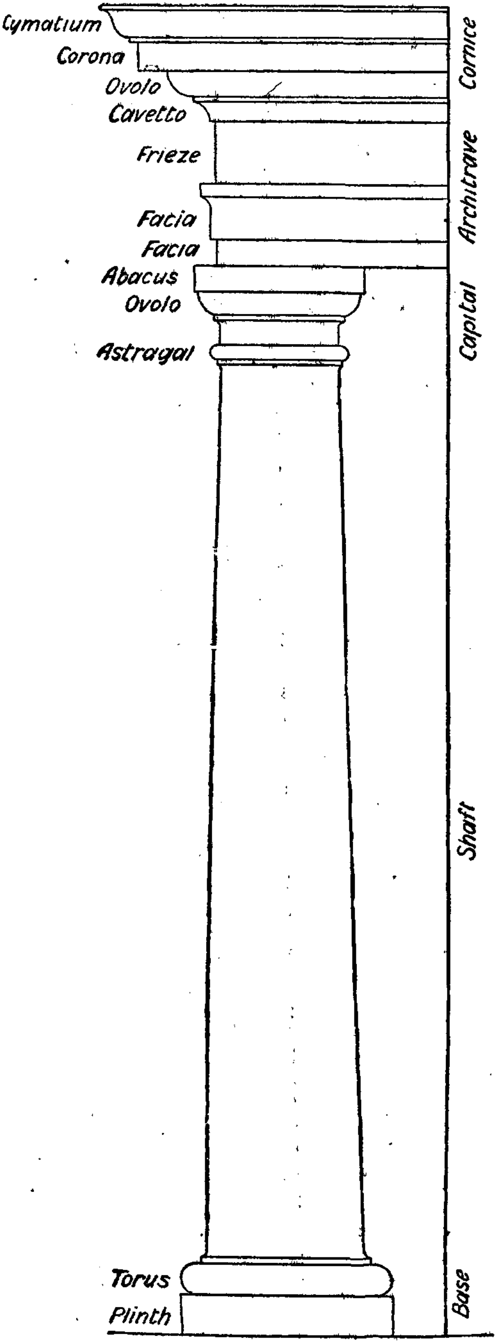

Cymatium
Corona
Ovolo
Cavetto
Frieze
Facia
Facia
Abacus
Ovolo
Astragal
Torus
Plinth
Cornice
Architrave
Capital
Shaft
Base

Fig. 1912.—Tuscan Order

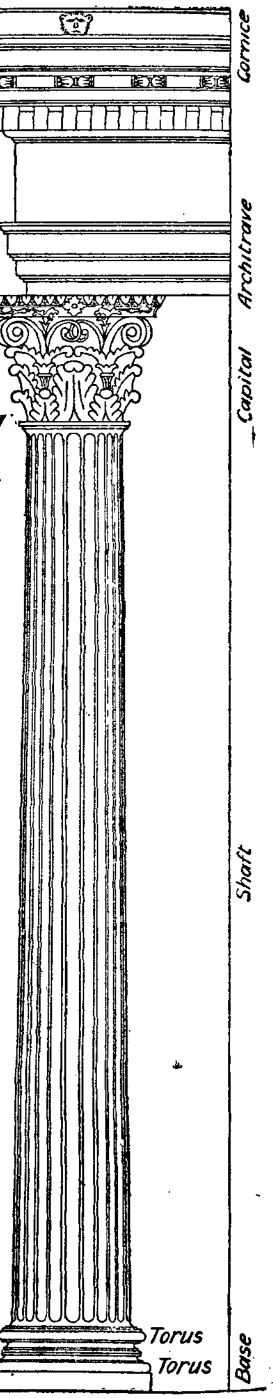

Cymatium
Corona
Modillions
Dentils
Frieze
Facia
Abacus
Caulicoli
Astragal
Scotia
Plinth
Torus
Torus
Cornice
Architrave
Capital
Shaft
Base

Fig. 1913.—Corinthian Order.

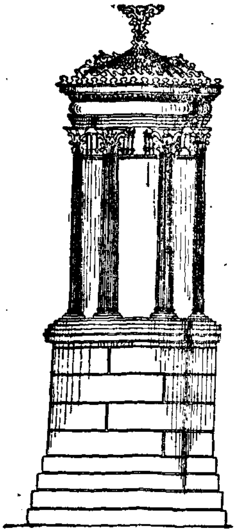


Fig. 1914.—Monument of Lysicrates.

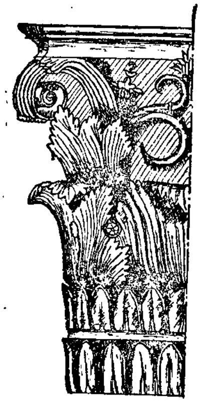


Fig. 1915.

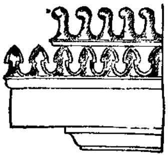


Fig. 1917.

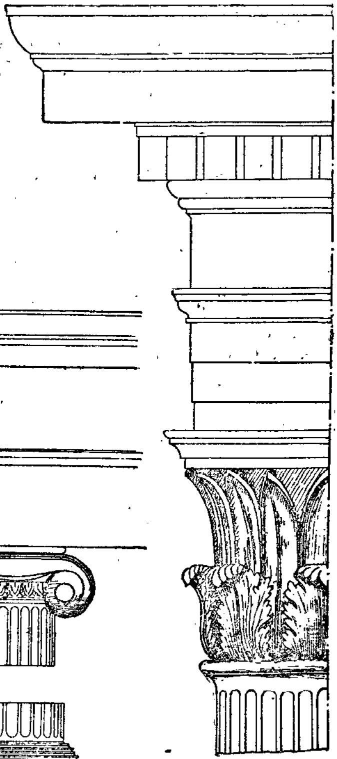


Fig. 1919.

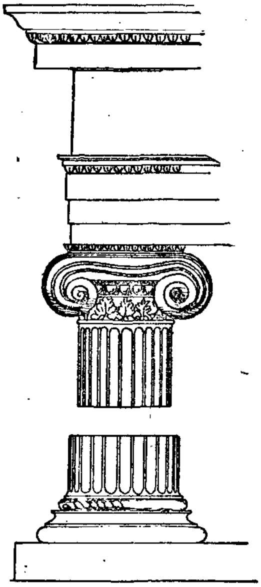


Fig. 1920.

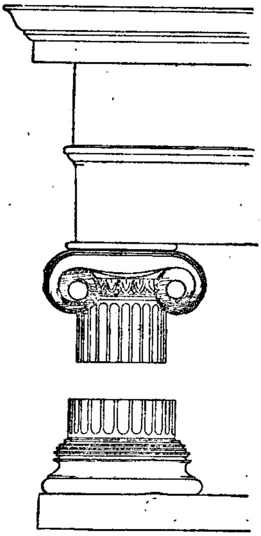


Fig. 1921.

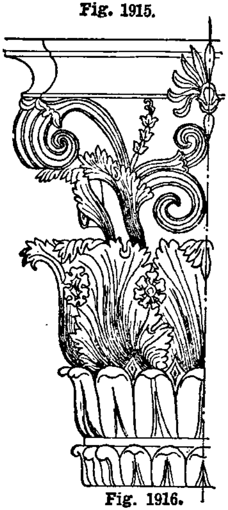
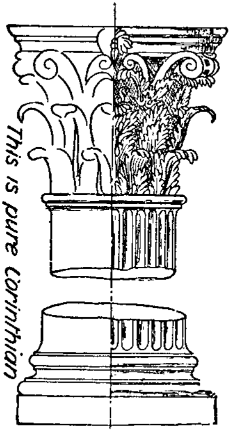


Fig. 1916.



This is pure Corinthian for contrast

Fig. 1918.

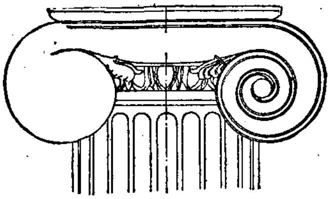


Fig. 1922.

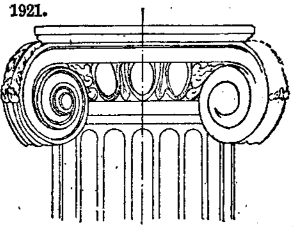


Fig. 1924.

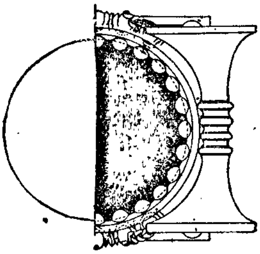


Fig. 1923.

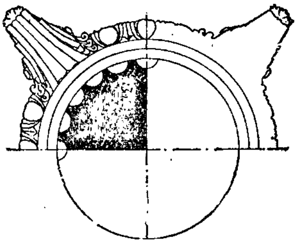


Fig. 1925.

Figs. 1915 to 1918.—Details of Monument of Lysicrates. Fig. 1919.—Capital and Entablature from Temple of the Winds. Figs. 1920 and 1921.—Greek Ionic Examples. Figs. 1922 to 1925.—Roman Ionic Examples.

Pæstum, three Grecian Doric temples, one hypæthral, one enneastyle, one hexastyle; at Syracuse, the Grecian Doric temple of Minerva; at Silenus, the temple of Jupiter in Grecian Doric hypæthral octastyle, and a smaller temple in Grecian Doric hexastyle: these places are all in Sicily. In Asia Minor there were—at Miletus, the Ionic temple of Apollo Didymæus; at Teos, the Ionic temple of Bacchus; at Priene, the Ionic temple of Minerva Polias; at Samos, the temple of Juno; at Ægesta, a Doric temple; at Ægina, the Doric temple of Jupiter Panhellenius. According to the Earl of Aberdeen ("Principles of Beauty in Grecian Architecture"), the temple of Minerva at Tegea was rebuilt by the famous Scopas of Paros in the concluding years of the Peloponnesian war, on the site of an older temple which was burnt down in the ninety-fourth Olympiad. This later temple was the largest, and, according to Pausanias, the most beautiful building in the Peloponnesus. It was an hypæthral edifice—that is, with the interior open to the sky. The interior of the cell was adorned with two rows of Ionic columns surmounted by others of the Corinthian order, being the first examples of the style in Greece mentioned by any writer. The peristyle was Doric, although Pausanias reverses the position of the Doric and the Ionic columns. A few shattered fragments constitute the only remains of this once magnificent structure; but as the situation is precisely ascertained, the whole plan might be easily restored.

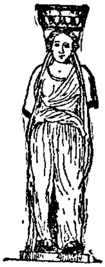


Fig. 1926.—Caryatide Figure.

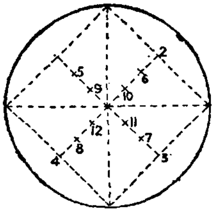


Fig. 1926a.—Eye of Ionic Volute.

Construction of Ionic Volute.

Fig. 1908, p. 507, shows an Ionic volute. To construct it, divide the height A B into eight equal parts, and number as shown in Fig. 1908 Draw horizontal centre line at 3½. Set off seven similar parts on base B C, and draw vertical centre line through E 3. The eye of spiral is set off as shown in Fig. 1926a, being one part in diameter. Commence with radius from point 1, and draw as far as centre line; then from point 2 reducing radius to continue from last portion, and so on. For the inner spiral, work backwards from where the double line commences, taking the centre at one-fourth the distance from 8 to 12 as shown by cross-marks on Fig. 1926a; continue with centre at one-fourth 7 to 11, and so on.

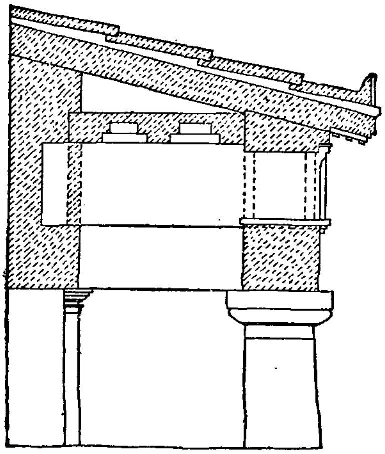


Fig. 1927.

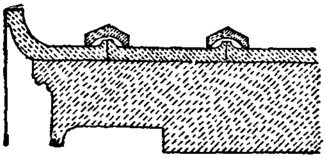


Fig. 1928.

Figs. 1927 and 1928.—Sections of Greek Roof Tiling.

Five Orders of Architecture.

What are known as the five orders of architecture comprise the Doric (Fig. 1909), the Ionic (Fig. 1910), the Composite (Fig. 1911), the Tuscan (Fig. 1912), and the Corinthian (Fig. 1913).

Three Examples of Greek Doric Order.

The best three examples of the Doric order are:—The temple at Corinth (B.C. 650), the temple of Theseus at Athens (B.C. 465), the

Parthenon at Athens (B.C. 438). The height of one of the columns of the Parthenon is 11 modules 0 parts (18 ft. 8·8 in.); the entablature, 4 modules 11 parts (7 ft. 1·75 in.). The height of a Doric column in general varies from 4 to 6½ of the lower diameters (8 to 13 modules). Other Doric examples are:—Temple of Minerva at Athens, temple of Jupiter Nemæus between Argos and Corinth, the Propylæa and Portico of the Agora at Athens, the temple of Apollo in Delos, the temples of Juno Lucina and Concord at Agrigentum, the temple at Egesta, the temples at Pæstum and Silenus.

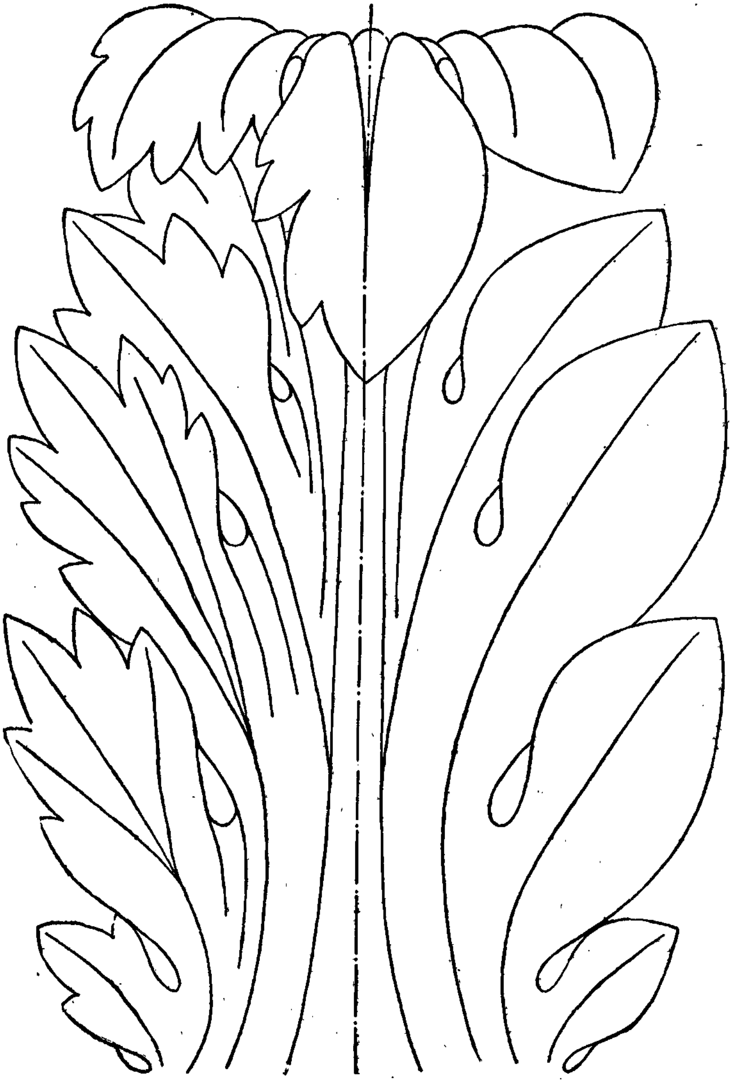


Fig. 1929.—Acanthus Leaf.

Choragic Monument of Lysicrates.

The Choragic Monument of Lysicrates, called also the Lantern of Demosthenes, is one of the most beautiful fragments of antiquity existing.

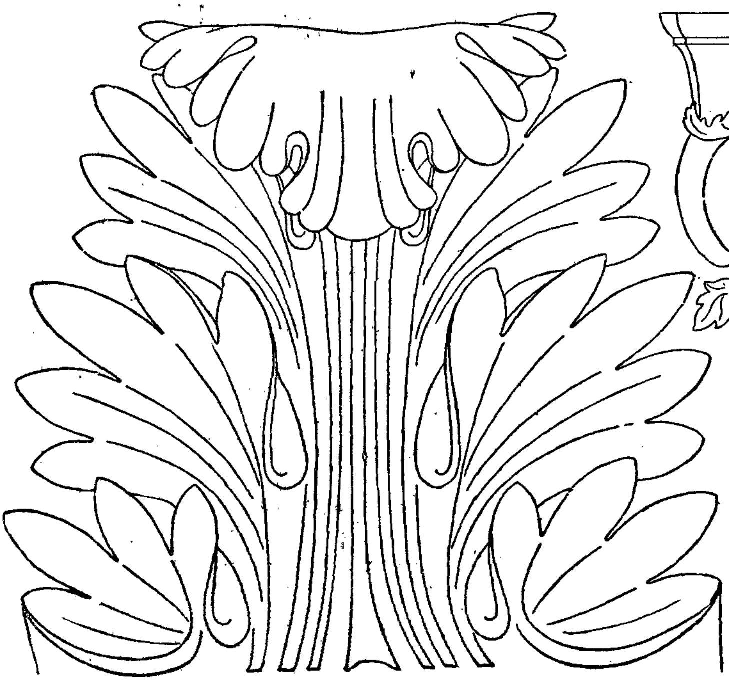


Fig. 1930.—Acanthus Leaf.

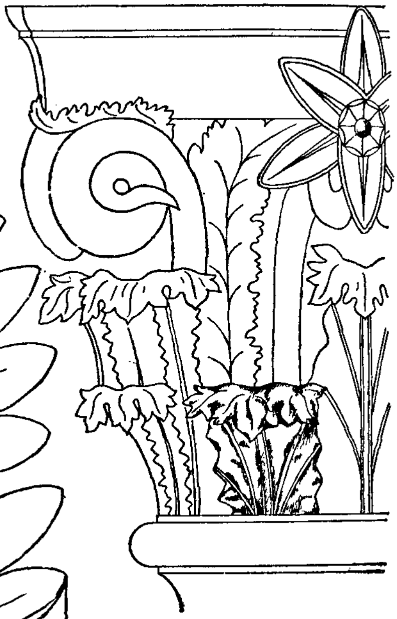


Fig. 1932.—Roman Corinthian Capital.

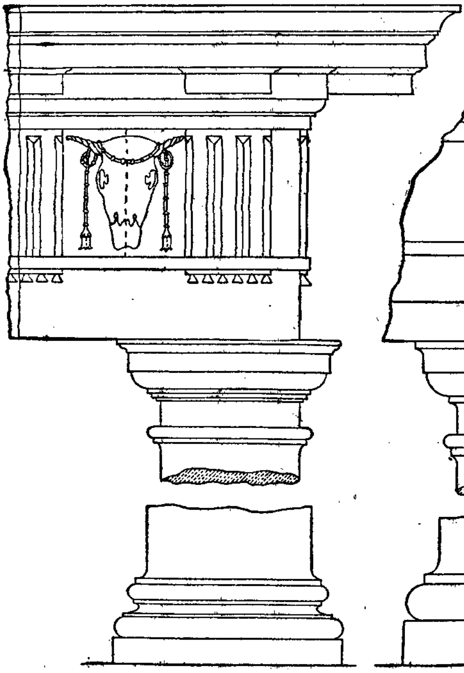


Fig. 1934.—Roman Doric Order.

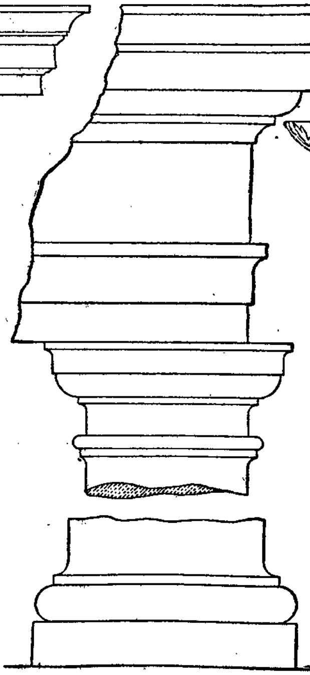
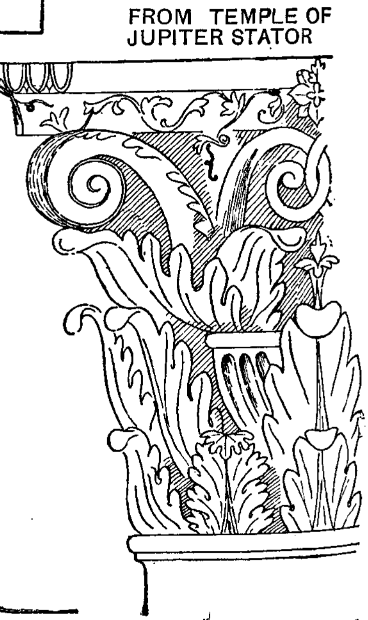


Fig. 1933.—Tuscan Order.



FROM TEMPLE OF JUPITER STATOR

Fig. 1931.—Roman Corinthian Capital.

The whole height is but 34 ft., and its diameter 8 ft. It is a circular temple, with six engaged columns standing on a basement nearly as high as the columns, and nearly solid. The capitals, though not like most Corinthian capitals, are very beautiful (see Figs. 1914 to 1918). The frieze is sculptured, and, instead of a cymatium to the cornice, there is an ornament of honeysuckles, and above that, on the roof, which is beautifully carved in leaves, is a line of waved projecting ornament; on the top is a vase, or more probably the base of a tripod.

Tower of the Winds.

The Tower of the Winds, or Temple of the Winds, also known as the octagon tower of Andronicus Cyrrhestes (see Fig. 1919), is chiefly valuable for its sculpture; it has two doorways of a composite order, and in the interior is a small order of a Doric of very inferior proportions, which rises to the support of the roof from a plain string, below which are two cornices, or rather tablets. The roof is of marble, cut into the appearance of tiles. The outside walls are plain, with an entablature and a string below, forming a sort of frieze, on which are the personified figures of the Winds. On the whole, this monument is curious rather than beautiful. (Rickman.)

Greek Ionic and Roman Ionic.

For the purposes of comparison, Figs. 1920 and 1921, showing the Greek Ionic order, and Figs. 1922 to 1925, showing the Roman Ionic order, are presented.

Caryatide Figures.

Fig. 1926 is a sketch of one of the Caryatide figures from the portico of the Erechtheum, Athens.

Greek Roof Tiling.

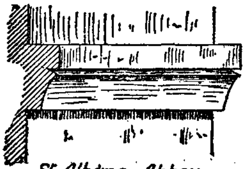
Figs. 1927 and 1928 are sectional views showing the mode of using roof tiles in Grecian architecture.

Acanthus Leaves.

Two varieties of acanthus leaf are illustrated by Figs. 1929 and 1930, the latter being the one used in the Corinthian order of architecture.

Openings in Greek and Roman Architecture.

In Grecian architecture the openings were of minor importance, and were necessarily small, owing to the trabeated (or beam) style of the architecture requiring the supports to be close enough to carry stone lintels of such length as could be obtained, and some of the openings were diminished towards the top. The monotony of a flat surface with rectangular openings was relieved by the heavy shadows in the spaces between the columns of the façade. In Roman architecture the openings were usually surmounted by semicircular arches, and the whole structure was treated with more freedom, the openings being made prominent features of the building. The round-headed arches of Norman architecture were due to the revival of Roman models.


S^{t} Albans Abbey
Fig. 1935. *c. 1080*

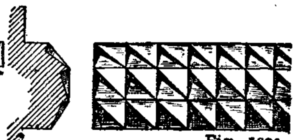
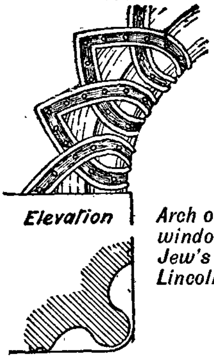


Fig. 1936.
Norman Chamfer
Westminster Hall, c. 1097


Elevation
Plan
Fig. 1937.
Arch over window
Jew's House
Lincoln 1140

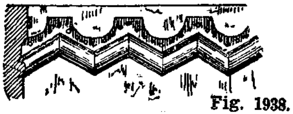
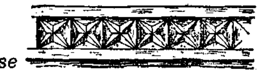
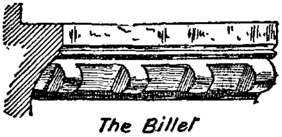
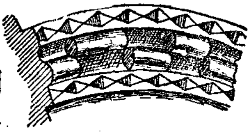
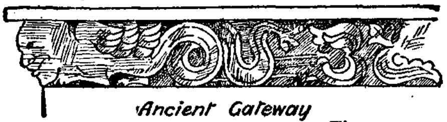


Fig. 1938.
Chevron, or Zig-zag with beads
Andover


The Star
Herringfleet, c. 1100
Fig. 1939.


The Billet
Malmesbury Abbey c. 1130
Fig. 1940.


The Billet & Lozenge
Walmer c. 1120
Fig. 1941.


Ancient Gateway
Lincoln 1150
Fig. 1942.

Figs. 1935 to 1942.—Norman Ornament.

Roman Arches.

The arch of Titus (A.D. 81) is a single arch of the Roman Composite order. The arch of Septimius Severus and the arch of Constantine each consist of a central arch with a smaller one on each side. All three of these are well known.

Roman Corinthian Capital.

Figs. 1931 and 1932 show in elevation two Roman Corinthian capitals. The outlines of the ornament are made clear, and only one leaf is drawn in detail. That shown by Fig. 1931 is from the temple of Jupiter Stator.

Tuscan and Roman Doric Orders.

Columns and entablature of the Tuscan and Roman Doric orders are shown in Figs. 1933 and 1934.

English Architecture Classified.

According to the simplest classification English architecture is divided into round arched and pointed arched. Some writers however, look upon Anglo-Saxon and Norman as distinct styles transplanted to these shores from abroad, and give the title of English architecture only to those styles developed in England—namely, those known as Early English, Decorated, and Perpendicular—assuming that the later developments were not worthy of the name of architecture. Others make a very full classification, enumerating Anglo-Saxon, Norman, Transition, Early English, Geometrical, Decorated, Flamboyant, Perpendicular, Elizabethan, Jacobean, Queen Anne, Adam architecture, etc.

Anglo-Saxon Architecture.

Anglo-Saxon architecture dates from the latter part of the seventh century to about the middle of the eleventh century. The buildings were mostly cruciform in plan, with a central tower and a circular apse at the eastern end; sometimes the western end was also circular, as at Abingdon, Berkshire, and a second tower was sometimes placed at the western end, as at Ramsey, in Huntingdonshire. The larger buildings had crypts with vaulted roofs, as at Repton, in Derbyshire. Sometimes a tower was placed only at the western end, as at Barton-on-Humber, in Lincolnshire, and at Brixworth and Earl's Barton, in Northamptonshire. The construction in many cases bore a strong resemblance to timber work, having a kind of framework of stone filled in with rubble; the quoins were typical of Anglo-Saxon style, being arranged with stones of equal size placed alternately in a vertical and horizontal position upon each other, called "long and short work." The heads of the doorways were either triangular or semicircular. The prevailing character is massiveness, with only the occasional introduction of a moulding, which in most cases consists simply of a square-faced projection with a chamfer or splay upon the upper or lower edge. The principal buildings of which remains exist are a church at Ripon, cathedral at Hexham, monastery at Wearmouth (A.D. 674), monastery at Jarrow (A.D. 684), church at Abingdon, cathedral at York (A.D. 767), abbey at Ramsey, Huntingdonshire (A.D. 974), church at Barton-on-Humber, Lincolnshire, churches at Brigsworth and Earl's Barton, Northamptonshire, church at Barnack, and, lastly, monastery at Bury St. Edmunds. Westminster Abbey was rebuilt A.D. 1065, and is in the Anglo-Norman style, although, as the date just given indicates, its reconstruction was planned and executed before the Conquest had taken place.

Local Diversities in English Architecture.

The physical geography or surface geology of England varies with great regularity from east to west, and with less regularity in other directions, the strata being inclined so that all geological epochs are exposed to view at the same time. On the east coast there are the recent deposits of sand, gravel, and clay, followed further inland by the upper and lower chalk with beds of flint, then limestone in abundance, and further on sandstone; while as the west coast is reached there are found old red sandstone, slate, and lastly granite. A practically similar distribution is found from south to north, but there are various minor breaks and local peculiarities. For building choice would naturally be made in the first instance of the materials found on the spot, to avoid the cost of transit, and the natives would be the most efficient workmen in that material; hence there would grow up a local style or fashion depending on these circumstances. So in London and the eastern counties there are found much brickwork and tiled roofing, owing

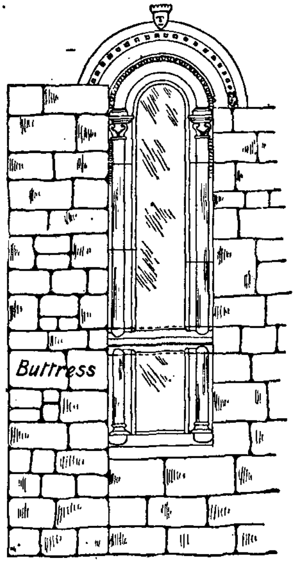


Fig. 1943.

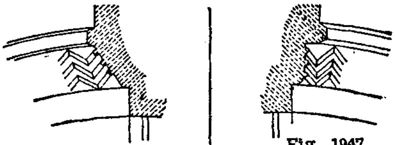


Fig. 1947.

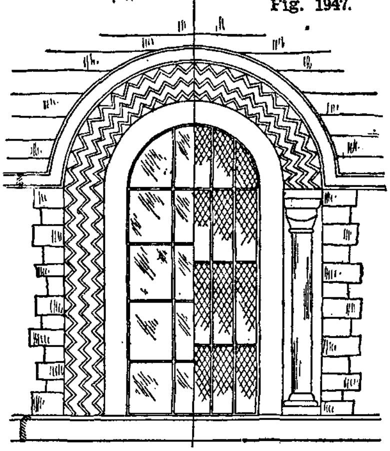


Fig. 1945.

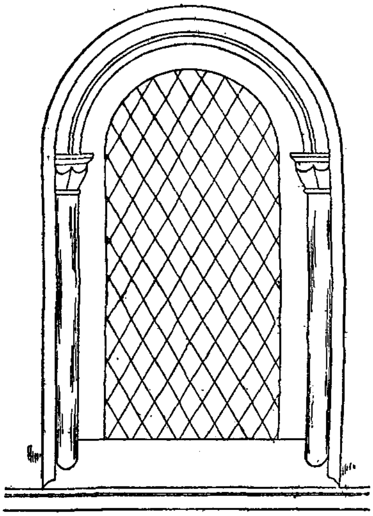


Fig. 1948.

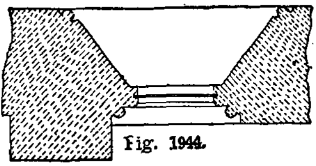


Fig. 1944.

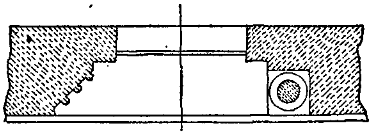


Fig. 1946.

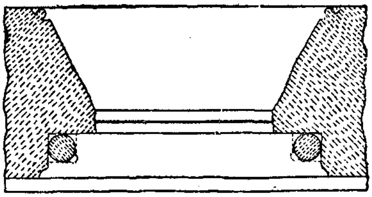


Fig. 1949.

Figs. 1943 to 1949.—Typical Norman Windows.

to the readiness with which clay is obtained; while in the West of England the very fences between the fields are made of granite, because of its abundance. It would be easy to show how the architectural details depend on the material in which they are worked, how the brickwork has to depend for its effect on the grouping of its masses, on the fenestration, the gables, and chimney stacks; how limestone lends itself to carving and moulding, so that Gothic cathedrals could be built from it; how granite, from its stubbornness, is only suited for heavy, massive work and classic architecture of great solidity. And it must not be forgotten that where forests abound, as formerly in England, half-timbered houses are the most natural structures, and may be found scattered over more than half the kingdom, from Ipswich to Chester and from Winchester to Lincoln. Other diversities would be produced by the local peculiarities of site and climate. Near the sea, hollow walls would be used to keep the interior of the building dry. In the open country, houses are built for roomy convenience and picturesqueness, generally on two or at most on three floors, and the appurtenances of the house proper are suitable to a country life. In a town, where land is more valuable, the rooms are piled on each other, and a cramped style of architecture is inevitable, although the design may be appropriate to the surroundings. Regarding the different periods of English architecture, it may be said that Anglo-Saxon terminated about 1066; Anglo-Norman flourished from 1066 to 1190; Early English, 1190 to 1300; Decorated English, 1300 to 1380; Perpendicular, 1380 to 1550; Elizabethan and Anglo-Italian, 1550 to 1625.

Romanesque Style.

Romanesque architecture is better known in England under the names of Saxon and

Norman architecture. It was introduced from Normandy by William the Conqueror and the Normans associated with him. It should therefore be called Norman-Romanesque. The year A.D. 1000 was expected to be the end of the world, and for some time previously there was very little building, and that only on a small scale; but the year having passed without any catastrophe, there was a general revival of building, and a return to substantial construction, cylindrical vaulting for aisle roofs, and, later, for nave roofs, being introduced. The models taken were chiefly the basilicæ of Rome and other cities of Italy, which had been erected between the time of Constantine, in A.D. 324, and the incursion of the Goths in the sixth century, but the style was largely influenced by the revival of Classic architecture in Italy and on the Continent generally. The Romanesque period in England may be taken as from 1000 to 1150 A.D. The chief points of the style were: thick stone walls without buttresses, semicircular arches in single rings of voussoirs, or in receding rings in thicker walls, piers and columns very massive and plain, capitals of columns in varieties of the cushion, windows only as narrow slits splayed on the inside with semicircular heads. Examples: Parts of Westminster Abbey, Waltham Abbey, Canterbury Cathedral, Rochester Cathedral, Chester Cathedral, St. Mary's, Walmer, St. John's, Chester, Studland Church, Chapel in the Tower of London, Hedingham Castle, Essex, etc.

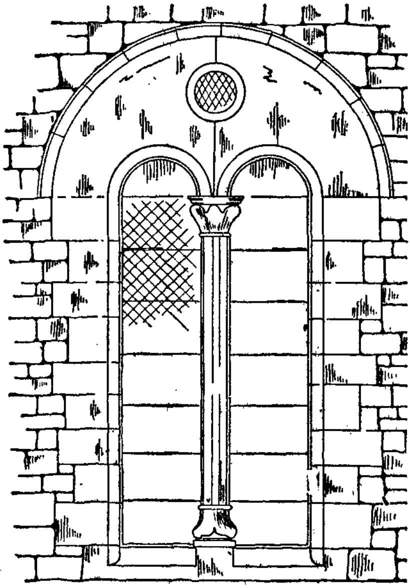


Fig. 1950.

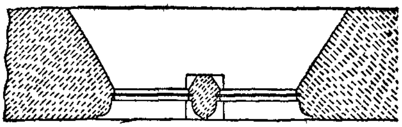


Fig. 1951.

Figs. 1950 and 1951.—Norman Window.

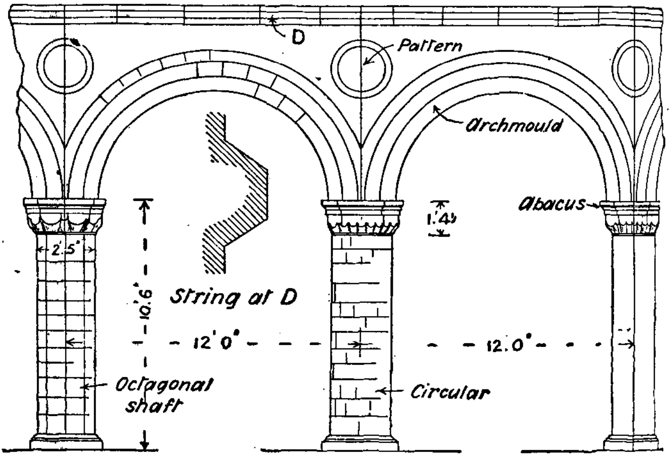


Fig. 1952.—Norman Nave Arcade.

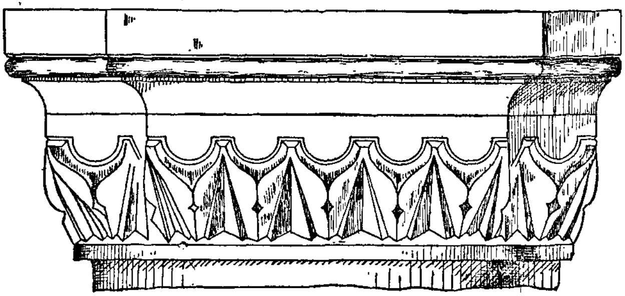


Fig. 1953.—Capital of Norman Column.

The Norman Styles.

Early Norman, 1066-1090. (*a*) Examples: White Tower, Tower of London; Transept, Winchester. (*b*) General style, massive and plain, with wide masonry joints. (*c*) Walls thick and plain, with sometimes shallow buttresses. (*d*) Doorways plain, round-headed with square angles, recessed and diminished in

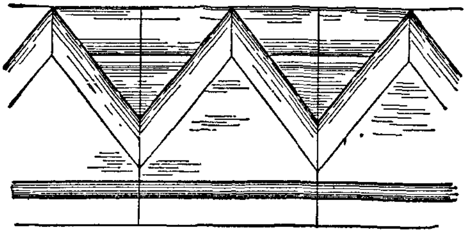


Fig. 1954.—Soffit at A (Fig. 1956).

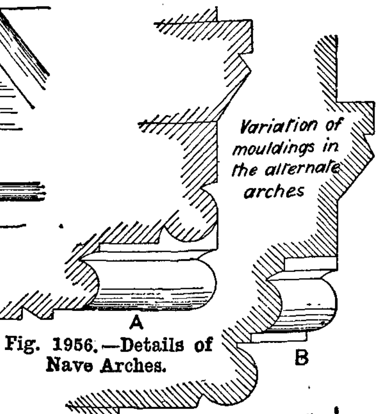

Variation of mouldings in the alternate arches
A
B

Fig. 1956.—Details of Nave Arches.

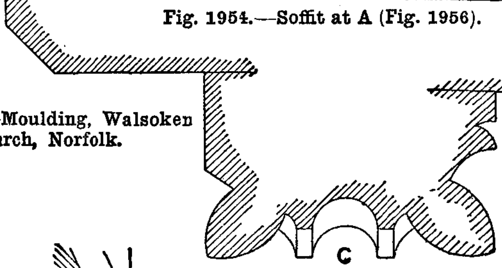


Fig. 1955.—Moulding, Walsoken Church, Norfolk.

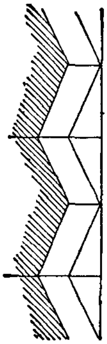


Fig. 1957.—Section at C (Fig. 1955).

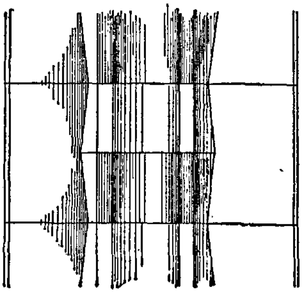


Fig. 1958.—Soffit at C (Fig. 1955).

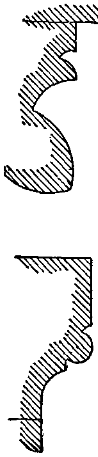


Fig. 1959.—Abacus.

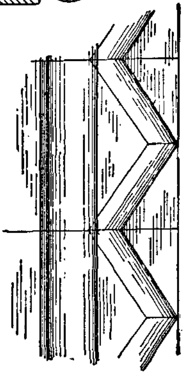


Fig. 1960.—Soffit at B (Fig. 1956).

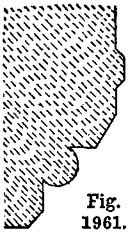


Fig. 1961. Norman Arch Moulding, Fountains Abbey.

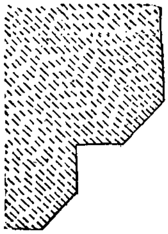


Fig. 1962.—Norman Arch Moulding, St. Peter's, Oxford.

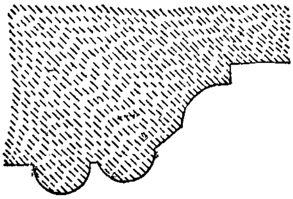


Fig. 1963.—Norman Arch Moulding, Stoke Orchard, Gloucester.

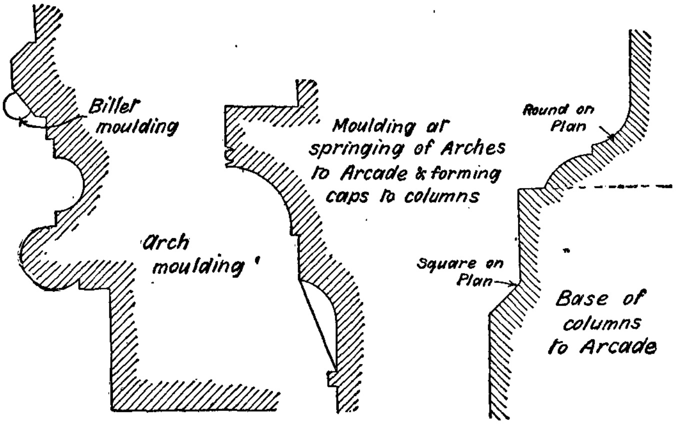

Billet moulding
Arch moulding
Moulding at springing of Arches to Arcade & forming caps to columns
Round on Plan
Square on Plan
Base of columns to Arcade

Fig. 1964.—Norman Arch Mouldings, Church of St. Bartholomew-the-Great, Smithfield.

thicker walls, no tympanum. (*e*) Mouldings were plain chamfer, round and quarter-round. (*f*) Windows long, narrow, and round-headed, in single lights or groups of single lights. (*g*) Arches all round-headed, plain square angles, recessed or doubly recessed. (*h*) Capitals to columns plain square block, with lower corners rounded off and the same style repeated side by side on the faces of larger blocks. (*i*) Vaulting narrow, plain semi-cylindrical shape, called barrel vaulting. (*j*) Ornament formed principally by the axe.

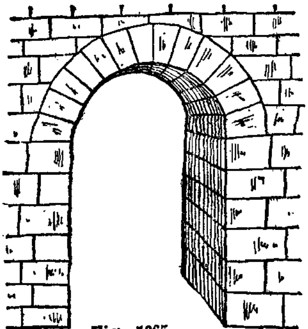


Fig. 1965.

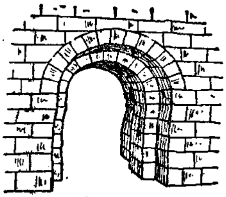


Fig. 1966.

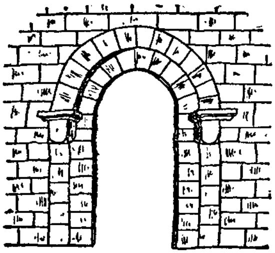


Fig. 1967.

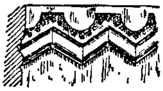


Fig. 1970.

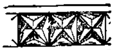


Fig. 1971.

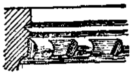


Fig. 1972.

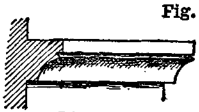


Fig. 1973.

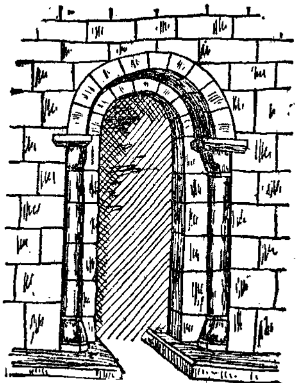


Fig. 1968.

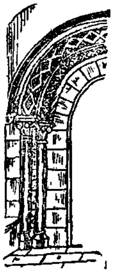


Fig. 1974.

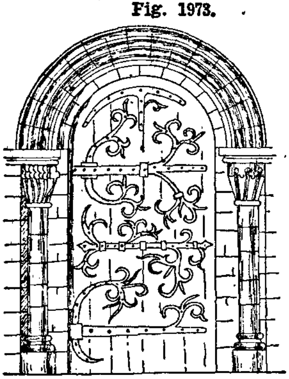


Fig. 1975.

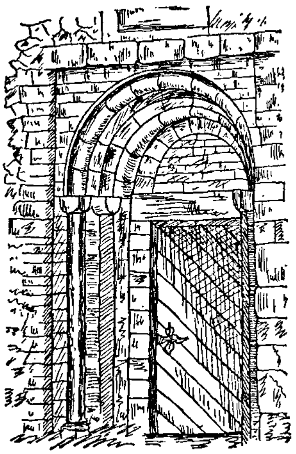


Fig. 1969.

Figs. 1965 to 1969.—Development of Norman Doorways. Figs. 1970 to 1974.—Norman Ornament. Fig. 1975.—Doorway at Faringdon, Berks.

Late Norman, 1090-1150. (*a*) Waltham Abbey; Peterborough Cathedral; Canterbury Cathedral; Iffley (Oxford); Barfreston (Kent); St. Martin's Priory (Dover); Newhaven Church (Sussex); Castor Church (Northants). (*b*) Structure lighter and abundantly ornamented. Stones dressed more truly and laid with thinner joints. (*c*) Walls thinner and buttresses more projecting, with small shafts recessed in the external angles. (*d*) Doorways round-headed and deeply recessed, giving splayed jambs and nook shafts. Tympanum richly sculptured. (*e*) Mouldings more numerous and deeper, but not in greater variety. Surfaces of mouldings ornamented. (*f*) Windows shorter and wider, still round-headed, but in two or three lights. Ornamented similar to the doorways. Large circular windows in the gable ends as at Waltham Abbey, Barfreston (Kent), and St. James's (Bristol). (*g*) Round-headed arches with deep edge mouldings and ornamented flat surfaces. Arcades introduced, followed by the first Pointed arches. (*h*) Capitals rudely carved with foliage, fruit, flowers, animals, figures, etc. The development may be traced in various examples. (*i*) Vaulting wider, groined, and sometimes ribbed. (*j*) Ornament worked with a chisel, and more undercut and elaborate. Eight specimens of Norman ornament are illustrated by Figs. 1935 to 1942. For plans

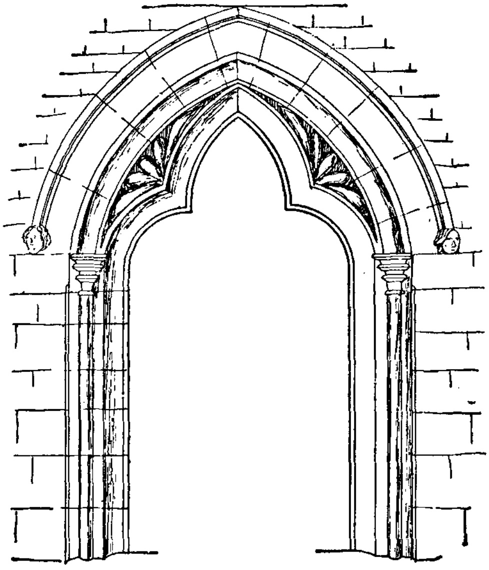


Fig. 1976.—Priests' Door, Irchester, Northants.

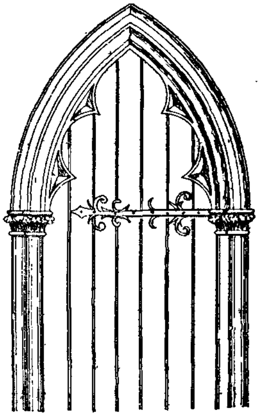


Fig. 1977. — Cinquefoil Arch.

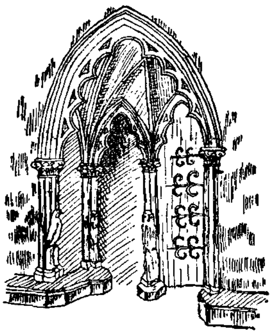


Fig. 1978.—Multiple Foiled Arch.

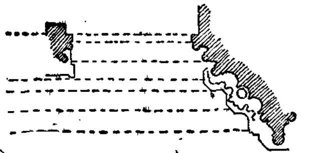


Fig. 1979.—Early English Door Mouldings.

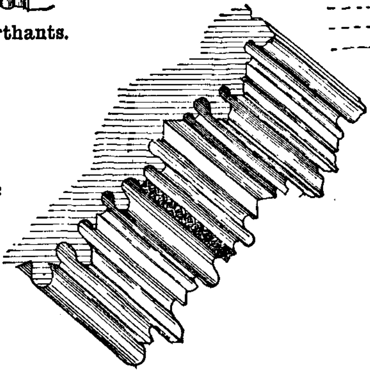


Fig. 1980.—Early English Door Moulding.

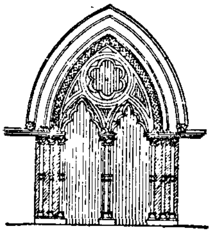


Fig. 1982. — Typical Early English External Doorway.

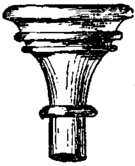


Fig. 1983. — Inverted Bell Capital.

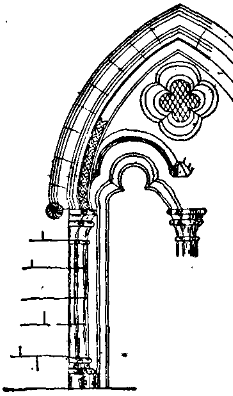


Fig. 1981.

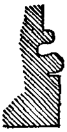


Fig. 1986.

Fig. 1981.—Typical Early English External Doorway. Fig. 1986.—Moulding at Base of Column.

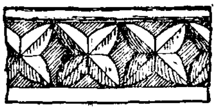


Fig. 1987.—Dog-tooth Ornament.

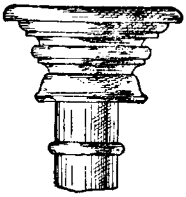


Fig. 1984.—Inverted Bell Capital.

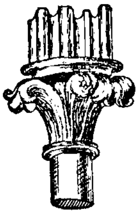


Fig. 1985. — Carving under Cap Moulding to Column.

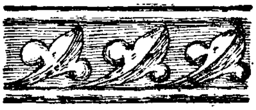


Fig. 1988.—Dog-tooth Ornament.

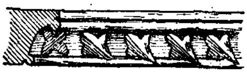


Fig. 1989.—Dog-tooth Ornament.

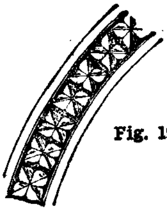


Fig. 1990.

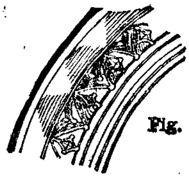


Fig. 1991.

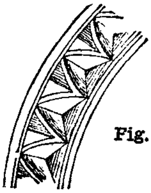


Fig. 1992.

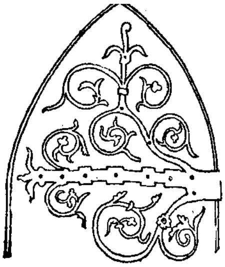


Fig. 1993.

Figs. 1990 to 1992.—Dog-tooth Ornaments.

Fig. 1993.—Door Hinge, St. Mary's Church, Norwich.

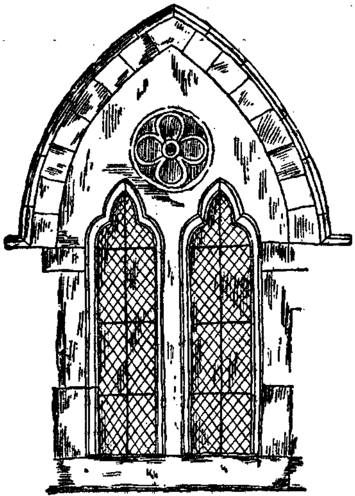


Fig. 1994.

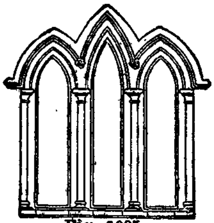


Fig. 1995.

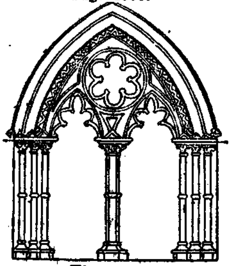


Fig. 1996.

Figs. 1994 to 1996.—Thirteenth-century Windows and Doorway.

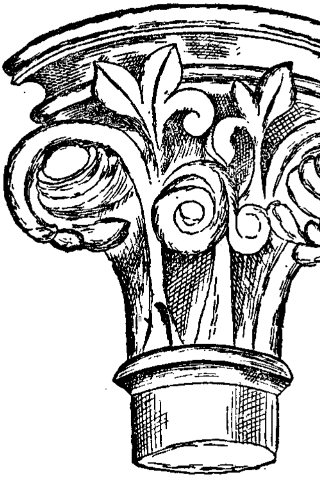


Fig. 1997.

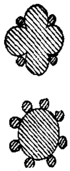


Fig. 1998.

Figs. 1997 and 1998.—Thirteenth-century Capital and Shafts.

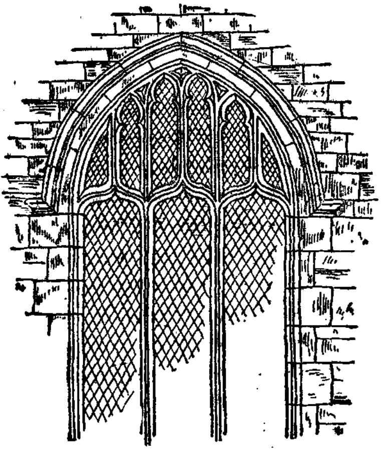


Fig. 1999.

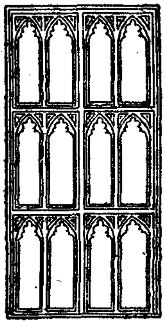


Fig. 2000.

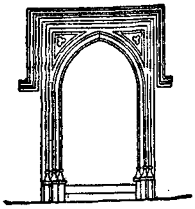
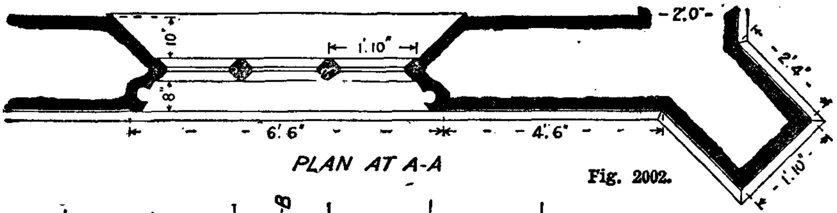


Fig. 2001.

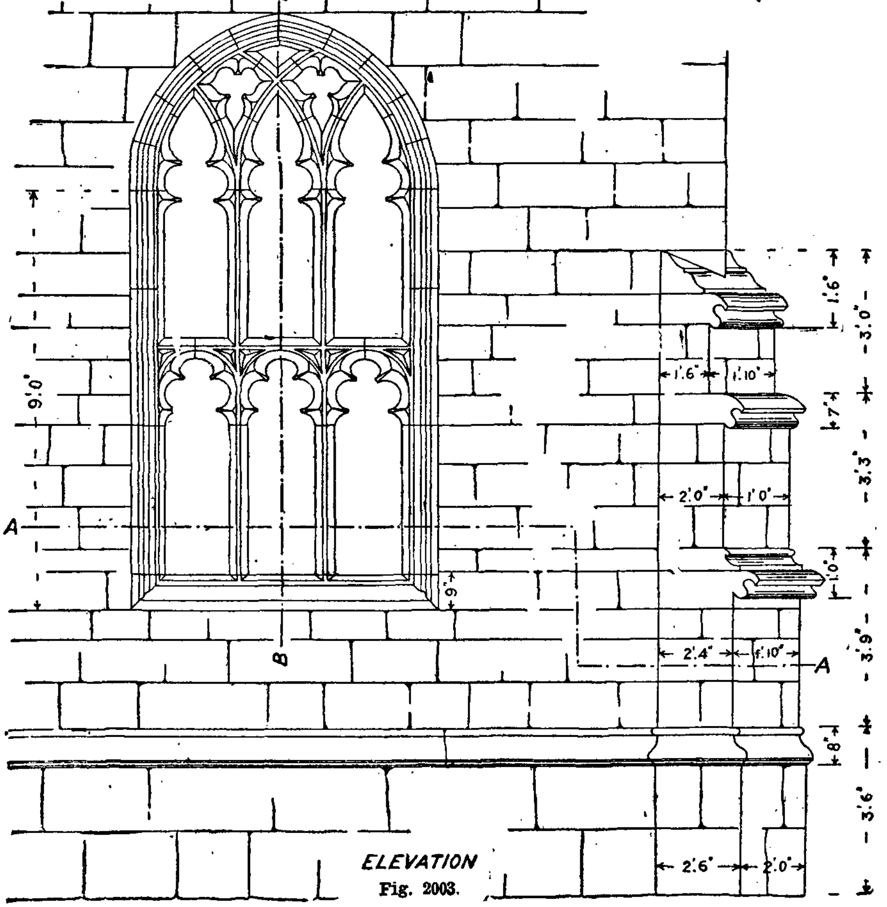
Figs. 1999 to 2001.—Fifteenth-century Windows and Doorway.

and elevations of typical Norman windows, see Figs. 1943 to 1951. Fig. 1952 represents the Norman nave arcade in Walsoken Church, Norfolk; Fig. 1953 a column capital; and Figs. 1954 to 1960 show details. Figs. 1961 to 1964 represent Norman arch mouldings from various buildings indicated, including St. Bartholomew-the-Great, Smithfield. The earliest Norman doorways were plain square-edged openings in the walls, with semicircular arches above (Fig. 1965). In thick walls the doorways were formed in the same manner, but in receding planes (Fig. 1966). Then cushion corbels were added (Fig. 1967), and these were followed by nook shafts with cushion capitals and bases (Fig. 1968). The square angles of the doorways then became rounded, with a quirk on each side forming a staff bead or the Norman



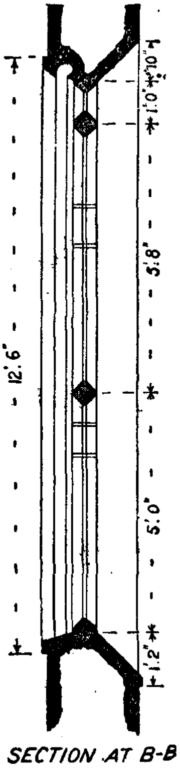
2'.0"
10"
1'.10"
8"
2'.4"
6'.6"
4'.6"
1'.10"
PLAN AT A-A

Fig. 2002.



B
9'.0"
A
9"
B
1'.6"
3'.0"
1'.6"
1'.10"
1'.7"
3'.3"
2'.0"
1'.0"
1'.0"
3'.9"
2'.4"
1'.10"
A
8"
3'.6"
2'.6"
2'.0"
ELEVATION

Fig. 2003.

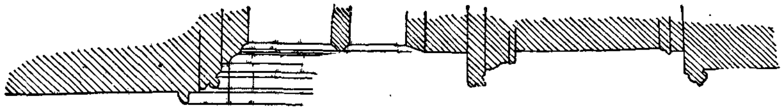


10"
1'.0"
5'.8"
12'.6"
5'.0"
1'.2"
SECTION AT B-B

Fig. 2004.

Figs. 2002 to 2004.—Sectional Plan, Elevation, and Vertical Section of Fourteenth-century Window.

edge roll or "bowtell" (Fig. 1969), and the surfaces between the mouldings were ornamented with various patterns, in which the zigzag or chevron (Fig. 1970) and the star (Fig. 1971) predominated. The billet moulding (Fig. 1972) was also frequent, and the Norman chamfer (Fig. 1973), which arose out of the ornamented dripstone, but was likewise used as a horizontal ornament. Later in the period the ornamentation became more profuse and the details of the structures less massive, the capitals of the columns rudely carved (Fig. 1974), and the shafts sometimes ornamented with spiral or zigzag diaper patterns. Several pairs of columns in receding and diminishing openings were employed, and the whole became richer in effect. When the actual opening of the doorway was rectangular, the


Section Fig. 2008.

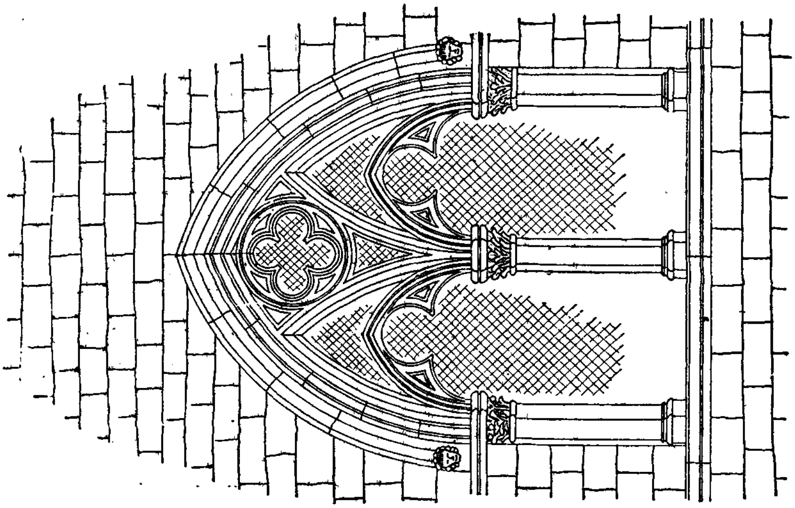


Fig. 2007. Elevation

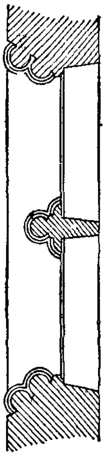


Fig. 2009. Plan

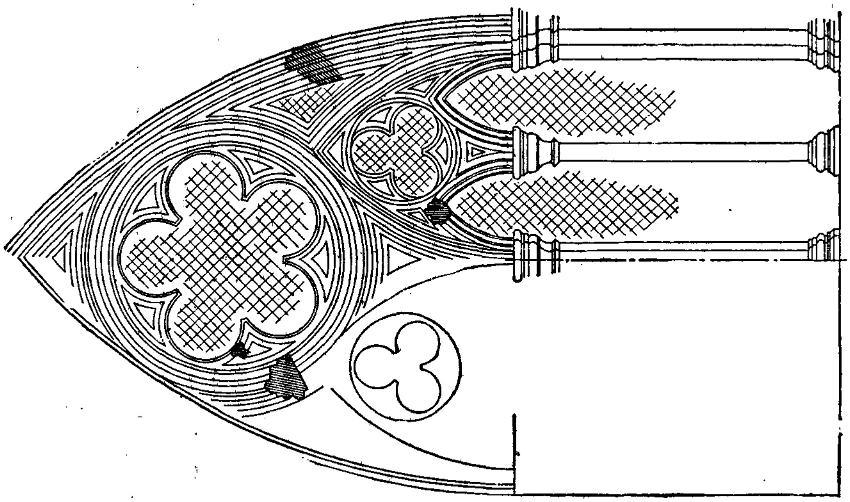


Fig. 2005. Elevation

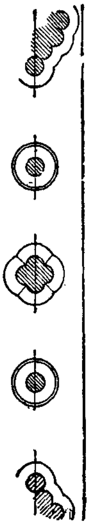


Fig. 2006. Plan

Figs. 2005 to 2009.—Early English Windows.

tympanum or space between the square head and semicircular arch was filled with sculpture, as at Rochester Cathedral and elsewhere.

Early English Doorways.

The doorway of the general Early English type is surmounted by an equilateral or wide lancet arch, with mouldings, but instances occur of a semicircular arch, as in Fig. 1975, at Faringdon, Berks; a trefoil arch, as in Fig. 1976, which represents the arch of the priests' door at Irchester, Northants; a cinquefoil arch, as in Fig. 1977; or a multiple foiled arch, as in Fig. 1978. The mouldings were

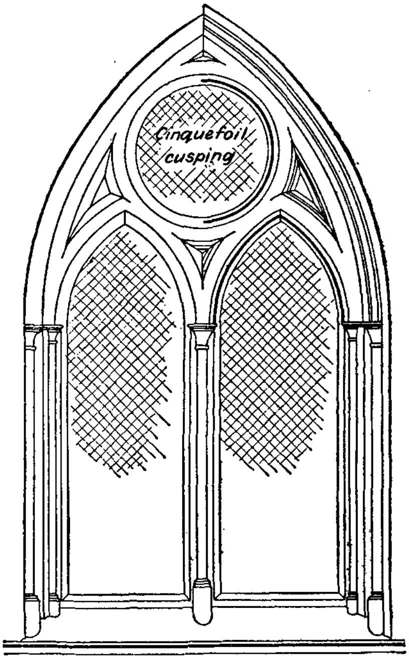


Fig. 2010. *Inside Elevation*

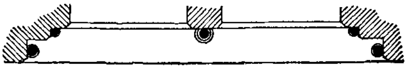


Fig. 2011. *Plan*

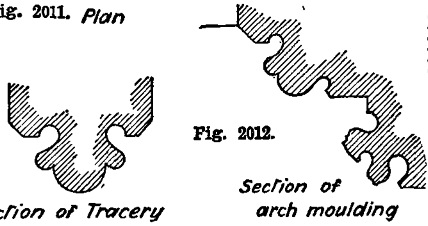


Fig. 2012.

Section of Tracery

Section of arch moulding

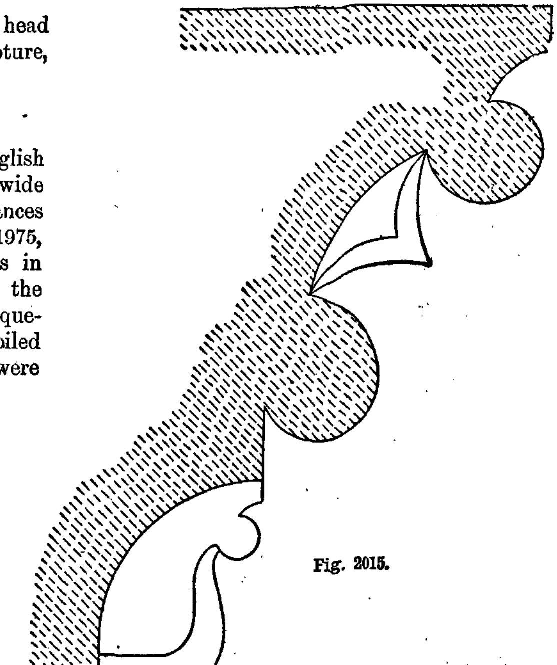


Fig. 2015.

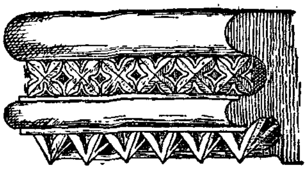


Fig. 2016.

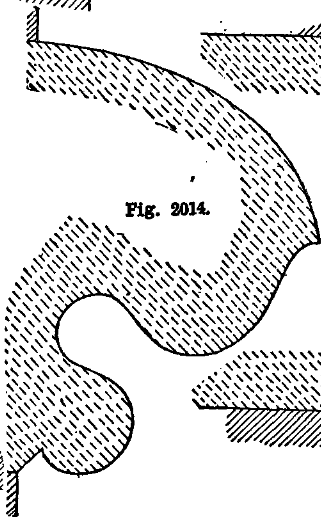


Fig. 2014.

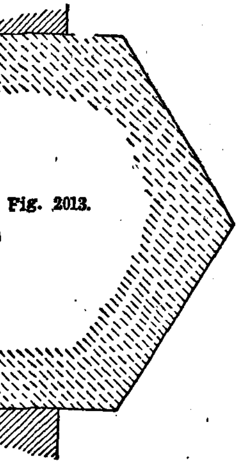
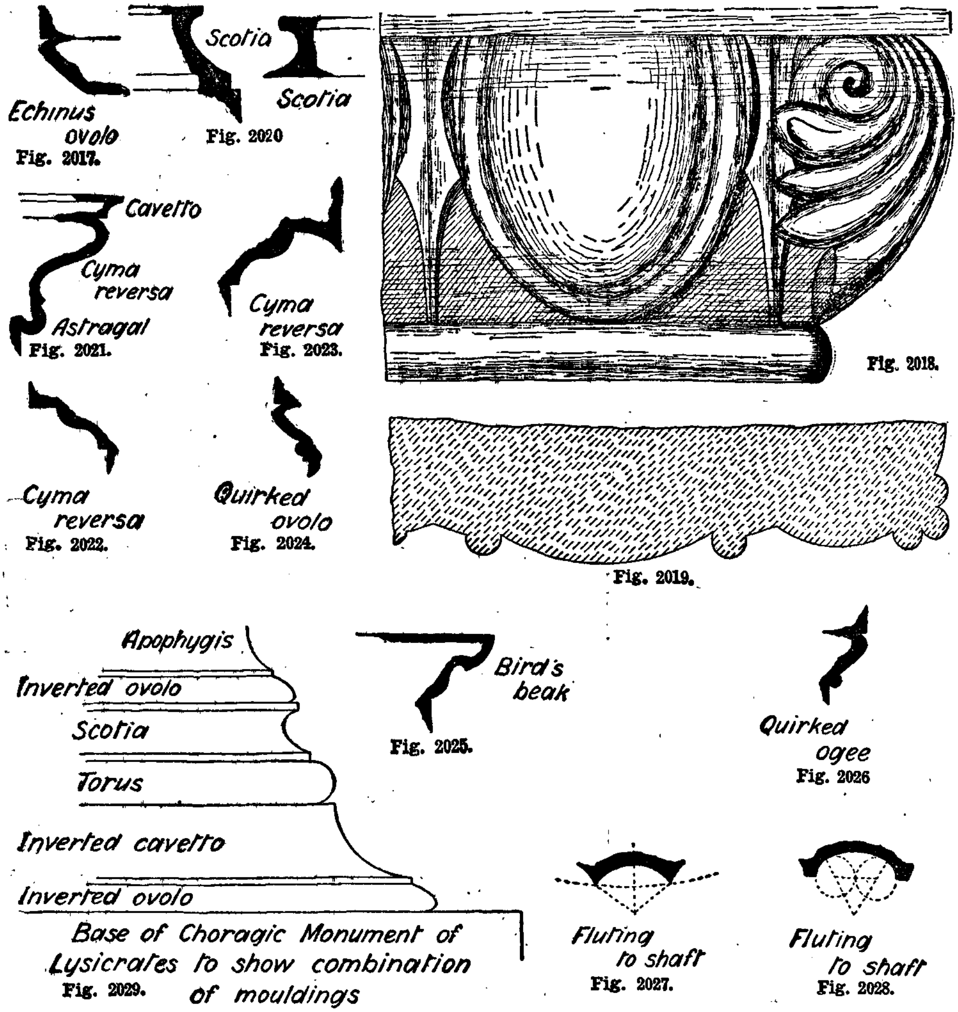


Fig. 2013.

Figs. 2010 to 2012.—Elevation, Plan, and Mouldings of Early English Window. **Figs. 2013 to 2016.—String-Courses.**

deep undercut hollows and rounds, with narrow fillets as in Figs. 1979 and 1980. Sometimes the door arches were without any foliation in their outline, and occasionally internal doorways were made with a square head The principal external doorways were divided by a central column, either a single shaft or a clustered column, the cylindrical portions being of polished marble; the tympanum or spandrel perforated with a trefoil, quatrefoil, or multiple-foiled opening, and the whole enclosed in a more or less deeply moulded and recessed arch with a dripstone on the face following the outline of the arch, and terminated in corbels moulded with heads or roses, as in Fig. 1981 (from St. Cross, Hampshire) and Fig. 1982. The side columns were mostly nook shafts in recessed jambs, with inverted bell capitals plainly moulded (Figs. 1983 and 1984), or enriched with leaves running up and curling over under the cap moulding (Fig. 1985). The bases of the column resembled the Grecian Attic base in having two rounds with a deep hollow between, as Fig. 1986. The dog-tooth ornament was very largely used for enriching the mouldings and


Echinus ovolo
Fig. 2017.
Scotia
Scotia
Fig. 2020
Fig. 2018.
Cavetto
Cyma reversa
Astragal
Fig. 2021.
Cyma reversa
Fig. 2023.
Cyma reversa
Fig. 2022.
Quirked ovolo
Fig. 2024.
Fig. 2019.
Apophygis
Inverted ovolo
Scotia
Torus
Inverted cavetto
Inverted ovolo
Base of Choragic Monument of Lysicrates to show combination of mouldings
Fig. 2029.
Bird's beak
Fig. 2025.
Quirked ogee
Fig. 2026
Fluting to shaft
Fig. 2027.
Fluting to shaft
Fig. 2028.

Figs. 2017 to 2029.—Greek Mouldings.

surfaces between them. There are varieties (as in Figs. 1987 to 1992) apparently derived from the zigzag and star ornaments of the Norman period, and sometimes carried down the jambs. The hinges of the doors were very elaborately wrought in scrollwork spread over the outer face of the door, as shown in Fig. 1993 (at St. Mary's Church, Norwich), and as already illustrated in Fig. 1975 (Faringdon, Berks).

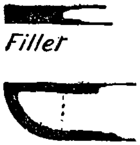


Fig. 2030.

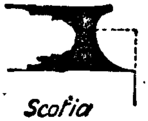


Fig. 2031.

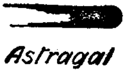


Fig. 2032.

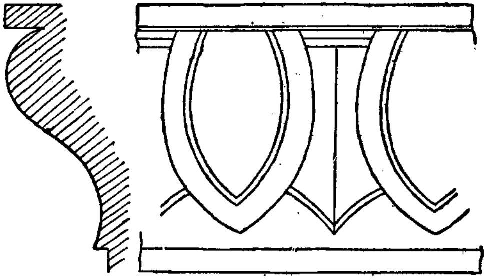


Fig. 2036.

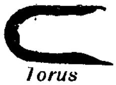


Fig. 2033.

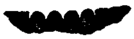


Fig. 2034.

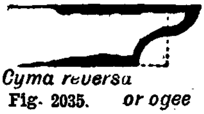

Cyma reversa
or ogee

Fig. 2035.

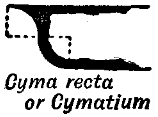

Cyma recta
or Cymatium

Fig. 2038.

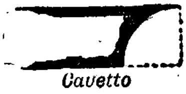


Fig. 2039.

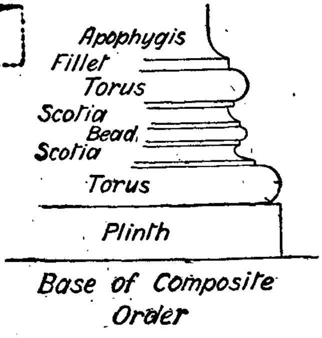

Apophygis
Fillet
Torus
Scotia
Bead
Scotia
Torus
Plinth
Base of Composite
Order

Fig. 2042.

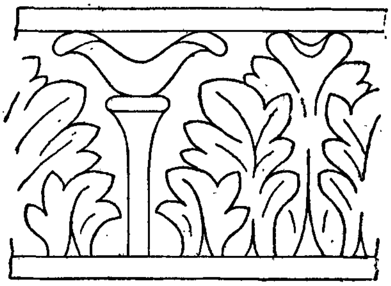


Fig. 2037.

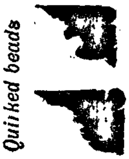

Quirked beads

Fig. 2040.

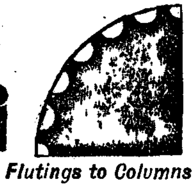

Flutings to Columns

Fig. 2041.

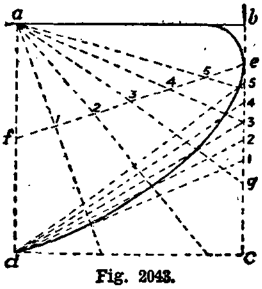

a
b
e
5
4
3
2
1
f
g
d
c

Fig. 2043.

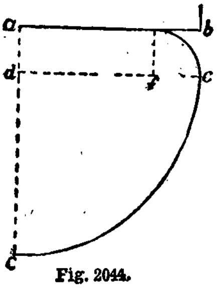


Fig. 2044.

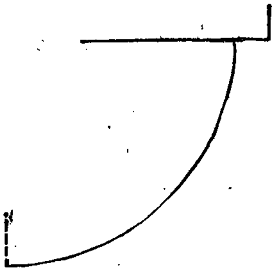


Fig. 2045.

Figs. 2030 to 2042.—Roman Mouldings. Fig. 2043.—Greek Ovolo. Fig. 2044.—Roman Moulding resembling Greek Ovolo. Fig. 2045.—Roman Ovolo.

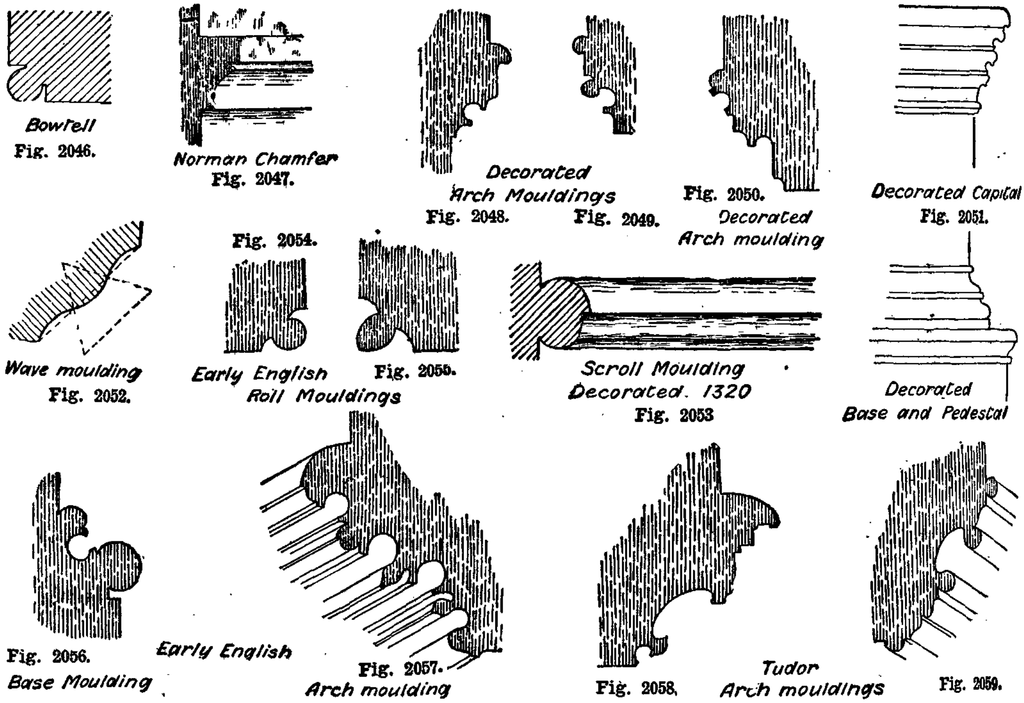
Thirteenth-century windows and a doorway are shown by Figs. 1994 to 1996, and capitals of the same period by Figs. 1997 and 1998. Fifteenth-century windows and a doorway are illustrated by Figs. 1999 to 2001.

Early English Windows and Base Mouldings.

Figs. 2002 to 2004 represent plan, elevation and section of a fourteenth-century window, taken from the south aisle, St. Peter's Church, Nottingham. Alternative sketches of Early English windows with their characteristic mouldings and other details are presented by Figs. 2005 and 2006, Figs. 2007 to 2009, and Figs. 2010 to 2012.

Early English String-courses.

Fig. 2013 shows the variety of string-course most frequently used in the Early English period, examples being in existence at Wrington, Somersetshire; Elsfield Church, Oxfordshire; and Ravensthorpe, Northants. Figs. 2014 and 2015 are other examples of Early English string-courses. An ornamental string-course of the thirteenth century is shown by Fig. 2016.


Bowtell
Fig. 2046.
Norman Chamfer
Fig. 2047.
Decorated
Arch Mouldings
Fig. 2048.
Fig. 2049.
Fig. 2050.
Decorated
Arch moulding
Decorated Capital
Fig. 2051.
Wave moulding
Fig. 2052.
Fig. 2054.
Early English
Roll Mouldings
Fig. 2055.
Scroll Moulding
Decorated. 1320
Fig. 2053
Decorated
Base and Pedestal
Fig. 2056.
Base Moulding
Early English
Fig. 2057.
Arch moulding
Fig. 2058.
Tudor
Arch mouldings
Fig. 2059.

Figs. 2046 to 2059.—English Mouldings.

Mouldings.

Greek Mouldings.—Echinus ovolo (Fig. 2017), which may have carved ornamentation as in Fig. 2018, scotia (Figs. 2019 and 2020), cavetto, cyma reversa, and astragal (Figs. 2021 to 2023), quirked ovolo (Fig. 2024), bird's beak (Fig. 2025), quirked ogee (Fig. 2026), fluting to shaft (Figs. 2027 and 2028), base of Choragic Monument of Lysicrates showing combination of mouldings (Fig. 2029).

Roman Mouldings.—Fillet (Fig. 2030), scotia (Fig. 2031), astragal (Fig. 2032), torus (Fig. 2033), reedings (Fig. 2034), cyma reversa or ogee (Fig. 2035) which may have carved ornamentation as in Figs. 2036 and 2037; cyma recta or cymatium (Fig. 2038), cavetto (Fig. 2039), quirked beads (Fig. 2040), flutings to columns (Fig. 2041), and base of composite order showing combination of mouldings (Fig. 2042).

Greek and Roman Mouldings Compared.—Regarding the difference between the Greek and Roman mouldings, Grecian ovolo (Fig. 2043) is not a regular compass curve, but partakes of the nature of some of the conic sections. In the example shown, *a b* indicates the projection of the curve, *b c* the depth. Complete the square *a b c d*, take point *e* one-sixth of *b c*, and *f* midway between *a* and *d*. Join *e f* and divide it into six equal parts. From *a* draw lines through each of these divisions *f*, 1, 2, 3, 4, 5, *e*, to meet the outline of the square. Take $e\,g = \frac{1}{2}\,b\,c$, and divide it into six equal parts, numbered from *g* upwards. From these points draw lines to meet point *d*, then the intersection of similarly numbered lines will give the outline of the Grecian quirked ovolo. A similar Roman moulding would be drawn with

ABCDEFGHIJKLMNOPQRSTUVWXYZ&c

ABCDEFGHIJKLMNOPQRSTUVWXYZ

Quantities

abcdefghijklmnopqrstuvwxyz

abcdefghijklmnopqrstuvwxyz

Specification

ABCDEFGHIJKLMNOPQRSTUVWXYZ&c

ABCDEFGHIJKLMNOP
QRSTUVWXYZ

Estimate

SHEET № DRAWING №

ABCDEFGHIJKLMNOP
QRSTUVWXYZ&c

1234567890

ABCDEFGHIJKLMNOPQRSTUVWXYZ&

Drawing №

1234567890

ABCDEFGHIJKLMNOPQRSTUVWXYZ&

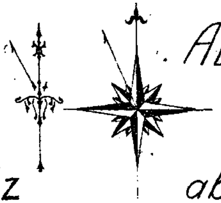
Scale. Scale.

SCALE. 1 INCH TO ONE FOOT

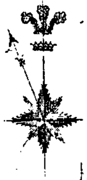
Scale of 1 in. to 1 ft.

Scale. 3 Feet to one Inch

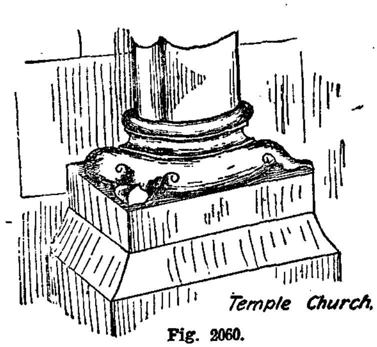
ABCDEFGHIJKLMNO
PQRSTUVWXYZ&
abcdefghijklmnopqrstuvwxyz



ABCDEFGHIJKLMNOPQR
STUVWXYZ&c
abcdefghijklmnopqrstuvwxyz



LETTERING FOR PLANS.



Temple Church, London

Fig. 2060.

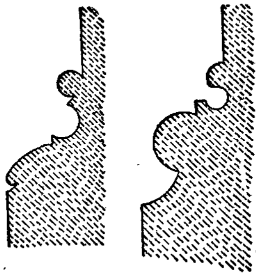
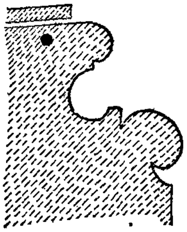
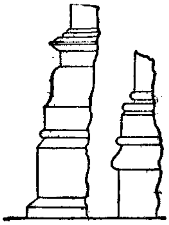


Fig. 2061.



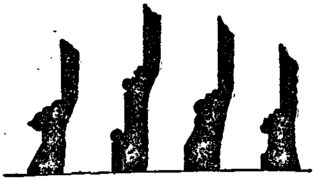
Stone Church, Kent

Fig. 2062.



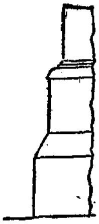
Fountains Abbey Choir

Fig. 2063.



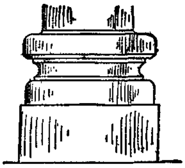
Tintern Abbey

Fig. 2064.



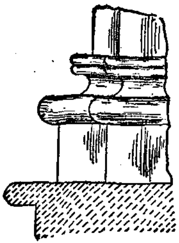
Howden Church

Fig. 2065.



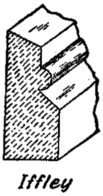
Rochester Cathedral

Fig. 2066.



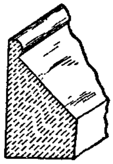
York Cathedral

Fig. 2067.



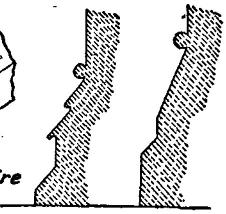
Iffley Church Oxfordshire

Fig. 2068.



Wolston Warwickshire

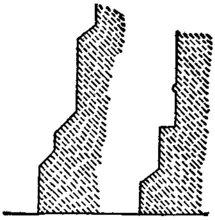
Fig. 2069.



Nave Choir

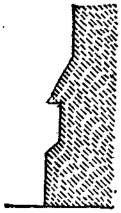
Tintern Abbey

Fig. 2070.



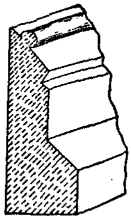
Fountains Abbey

Fig. 2071.



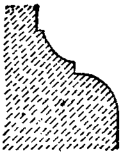
Howden Church

Fig. 2072.



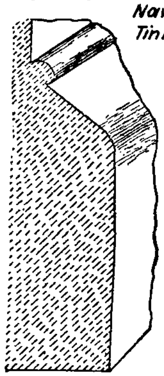
Steetley Ch. Derbyshire

Fig. 2073.



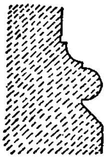
Peterboro A.D. 1143

Fig. 2074.



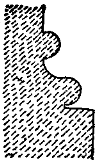
St Peters Northants

Fig. 2075.



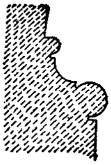
Glastonbury A.D. 1180

Fig. 2076.



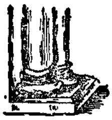
Castle Rising Norfolk

Fig. 2077.



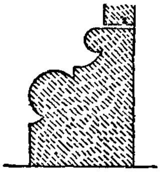
Peterboro'

Fig. 2078.



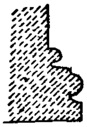
Window Shaft Stanton Harcourt

Fig. 2079.



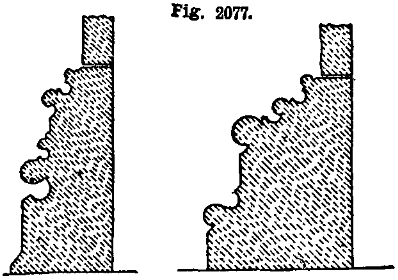
Casterton Church Rutland

Fig. 2080.



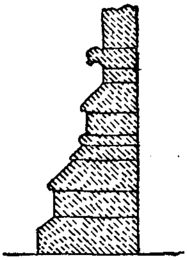
Polebrook Northants.

Fig. 2081.



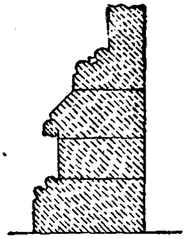
Lincoln Cathedral. A.D. 1220

Fig. 2082.



Hexham

Fig. 2083.



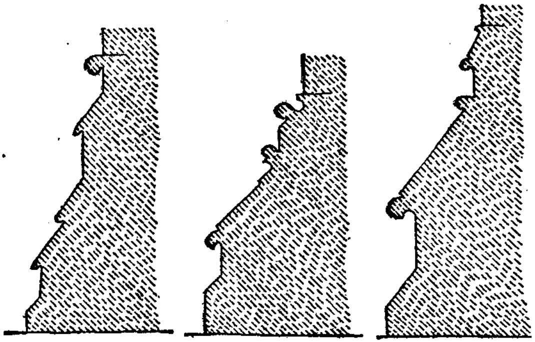
Bridlington

Fig. 2084.

Figs. 2060 to 2078.—Twelfth-century Mouldings. Figs. 2079 to 2084.—Thirteenth-century Mouldings.

compass curves as in Fig. 2044, where the whole curve is composed of two quadrants. In a plain Roman ovolo (Fig. 2045) a single quadrant is used.

English Mouldings.—The bowtell, boutel, or beautel (Fig. 2046) was the first moulding introduced into English architecture. It was like a staff bead or angle bead, being a double quirked roll on the square edges of arches and other openings in masonry. It is also known as the Norman edge roll, and was applied to pillars as well as to arches. The Norman chamfer moulding is shown by Fig. 2047; mouldings of the Decorated style by Figs. 2048 to 2051; the wave moulding or swelled chamfer by Fig. 2052, this being a shallow moulding of the Decorated period, consisting of three tangent curves struck from the angles of an equilateral triangle whose base is parallel to the chamfer plane; the scroll moulding belonging to the Decorated style by Fig. 2053; the Early English mouldings by Figs. 2054 to 2057; the Tudor mouldings by Figs. 2058 and 2059; twelfth-century mouldings by Figs. 2060 to 2078; thirteenth-century mouldings by Figs. 2079 to 2091; and fifteenth-century mouldings by Figs. 2092 to 2095.

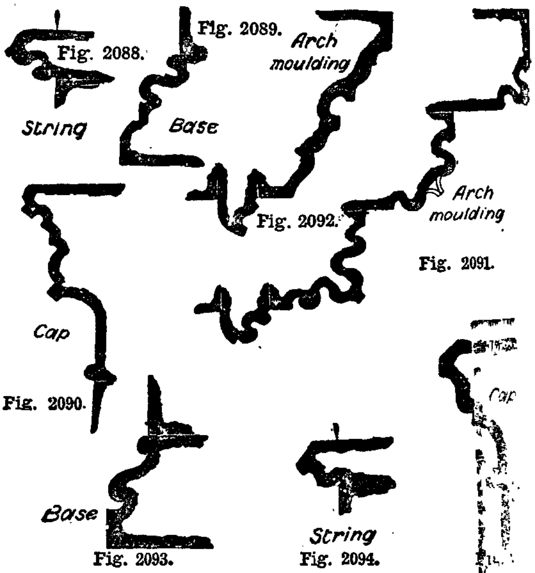


Bridlington Fig. 2085. *Beverley* Fig. 2086. *York* Fig. 2087.

Figs. 2085 to 2087.—Thirteenth-century Mouldings.

Development of the Abacus.

The Norman abacus was always square in plan (Fig. 2096). At first a plain rectangular block with the lower angles square in section (Fig. 2097), then with the lower edges chamfered (Fig. 2098), and the chamfer hollowed out as Fig. 2099, and finally rounded on top and hollowed below with a groove and fillet between, as Fig. 2100. The Early English abacus was circular in plan (Fig. 2101), and consisted at first of two round projecting mouldings with a deep hollow between, the upper roll being the larger, as Fig. 2102. Later in the period the mouldings were more numerous, and frequently with flat fillets on the face, as Fig. 2103. In the Decorated style the abacus was sometimes circular, but often octagonal in plan (Fig. 2104). When circular, the abacus simply formed the upper members of a series of mouldings, constituting the capital of the column, and the mouldings were all of the plain roll order (Fig. 2105), but less undercut than formerly, the scroll moulding being common. In the Perpendicular period the octagonal abacus was used, with moulded edges, as Fig. 2106; but in other cases, where circular, the abacus was not distinguishable from the mouldings of the capital of the column (see Fig. 2107). When the piers were clustered the capitals coalesced, and the mou ings ran continuously round the pier.



Figs. 2088 to 2091.—Thirteenth-century Mouldin
Figs. 2092 to 2095.—Fifteenth-century Moulding

Gothic Architecture.

The term Gothic was not used until t types had fallen into disuse, and was th introduced to signify barbarian (as Rome w overrun by the Goths and Vandals); a students of Classic architecture saw little beauty in pointed arches; but Ruskin did more than anyone else to rescue the spirit of Gothic architecture from oblivion by his powerful writing. In one paragraph he says: "The Gothic architecture arose in massy and mountainous strength; block heaved upon block by the monk's enthusiasm and the soldier's force, and cramped and stanchioned into such weight of grisly wall as might bury the anchoret in darkness and beat back the utmost storm of battle, suffering but by the

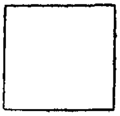


Fig. 2096.

Fig. 2097.

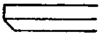


Fig. 2098.

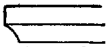


Fig. 2099.

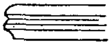


Fig. 2100.

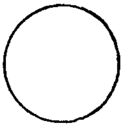


Fig. 2101.

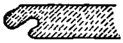


Fig. 2102.

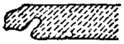


Fig. 2103.

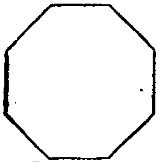


Fig. 2104.

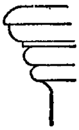
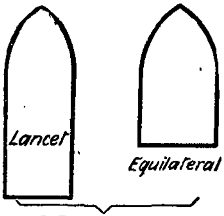
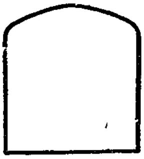
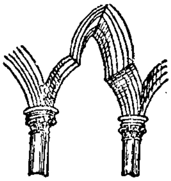


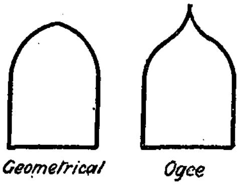
Fig. 2105



EARLY ENGLISH
12th. & 13th. Century
Fig. 2108.


Four centred or Tudor arch
PERPENDICULAR
15th Century
Fig. 2109


Early English
1195
Fig. 2110.



DECORATED
14th. Century
Fig. 2111.

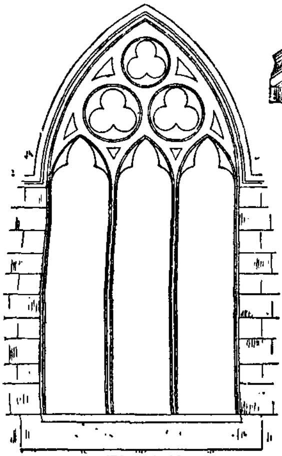


Fig. 2118.

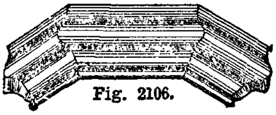


Fig. 2106.

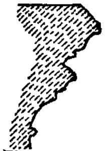


Fig. 2107.

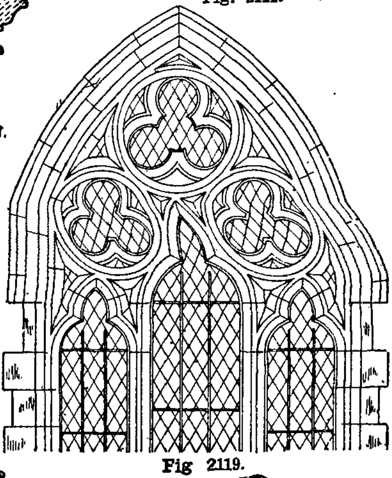


Fig 2119.

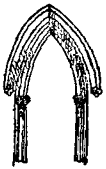


Fig 2112.

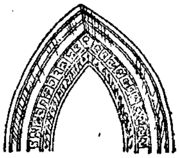
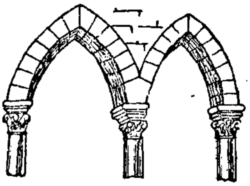
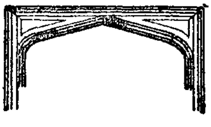
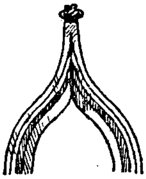


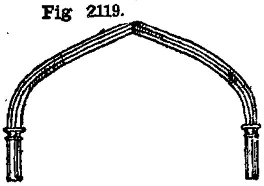
Fig. 2113.

Early English
1220 *1250*


Period of Transition
1178
Fig. 2114.


Perpendicular
1610
Fig. 2115.


Decorated. Ogee
1320
Fig. 2116.


Perpendicular
1435
Fig. 2117.

Figs. 2096 to 2100.—Development of Norman Abacus. Figs. 2101 to 2107.—Development of English Abacus. Figs. 2108 to 2117.—Development of Pointed Arch in England. Figs. 2118 and 2119.—Gothic Windows.

same narrow croslet the passing of the sunbeam or of the arrow. Gradually, as that monkish enthusiasm became more thoughtful, and as the sound of war became more and more intermittent, beyond the gates of the convent or of the keep, the stony pillar grew slender and the vaulted roof grew light, till they wreathed themselves into the semblance of the summer weeds at their fairest, and of the dead fieldflowers long trodden down in blood. Sweet monumental statues were set to bloom for ever beneath the porch of the temple or the canopy of the tomb." And later summarising the matter, he says: "We have seen that exactly in the degree in which Greek and Roman architecture is lifeless, unprofitable, and un-Christian, in that same degree our own ancient Gothic is animated, serviceable, and faithful. We have seen that it is flexible to all duty, enduring to all time, instructive to all hearts, honourable and holy in all offices. It is capable alike of all loveliness and all dignity; fit alike for cottage porch or castle gateway; in domestic service familiar, in religious sublime, simple and playful so that childhood may read it; yet clothed with a power that can awe the mightiest and exalt the loftiest of human spirits; an architecture that kindles every faculty in its workman, and addresses every emotion in its beholder; which, with every stone that is laid on its solemn walls, raises some human heart a step nearer heaven, and which from its birth has been incorporated with the existence, and in all its forms is, symbolical of the faith of Christianity." Welby Pugin, Sir Gilbert Scott, and other architects who have now passed away, helped considerably to foster the taste for Gothic architecture, and it would be difficult to say to whom the greatest credit was due.

Gothic Arches and Windows.

A number of sketches will now be given showing the progressive changes through which the pointed arch passed, from the rise to the fall of Gothic architecture in England (see Figs. 2108 to 2117). The pointed arch is said to have arisen from a constructional necessity, owing to the tendency of large semicircular arches to sink at the crown by the spreading of the abutments; for example, Furness Abbey. Figs. 2118 and 2119 show alternative designs for a Gothic window, the upper part being filled in with trefoil tracery. Figs. 2120 to 2129 are sketches of arch mouldings of the thirteenth, fourteenth, and fifteenth centuries respectively.

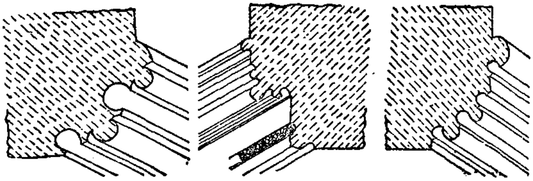


Fig. 2120. Fig. 2121. Fig. 2122.

13th century arch mouldings.

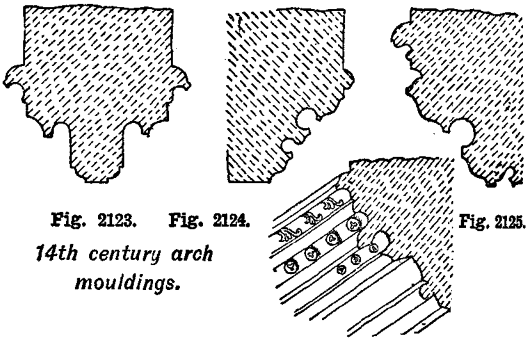


Fig. 2123. Fig. 2124. Fig. 2125.

14th century arch mouldings.

Fig. 2126.

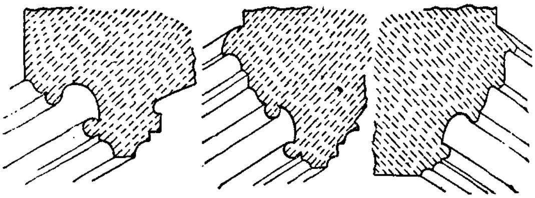


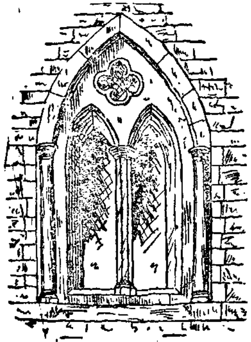
Fig. 2127. Fig. 2128. Fig. 2129.

15th century arch mouldings.

Figs. 2120 to 2129.—Arch Mouldings.

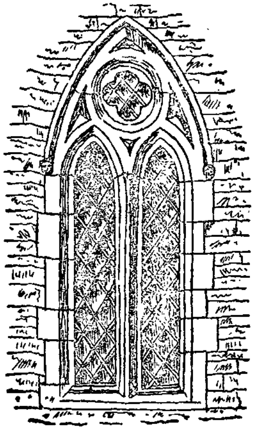
Plate and Bar Tracery.

Figs. 2130 to 2132 are sketches of plate tracery, and Figs. 2133 to 2140 of bar tracery. When two lights were gathered under one dripstone, a blank space, known as the tympanum, was left; but in process of time this space began to be pierced with another small light, in the form of an ellipse, a circle, or a trefoil, which at once relieved the blank space



c. 1250

Fig. 2130.



c. 1260

Fig. 2131.

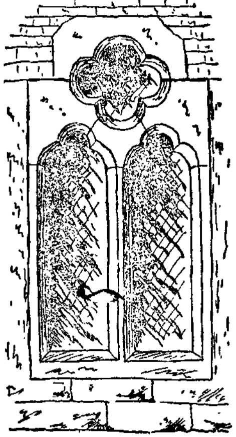
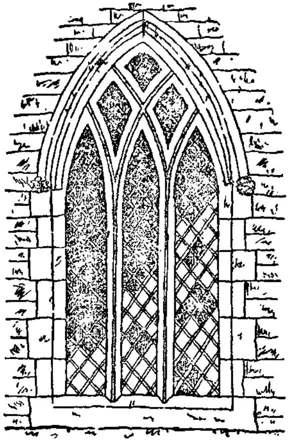
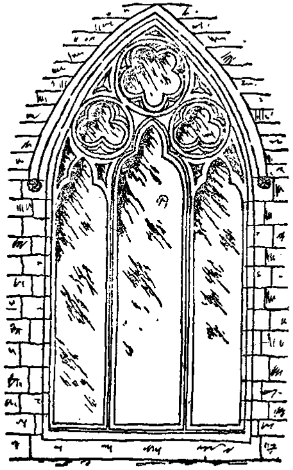


Fig. 2132.



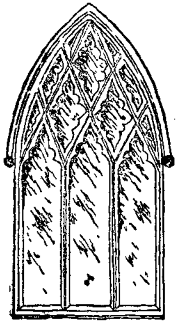
c 1260

Fig. 2133.



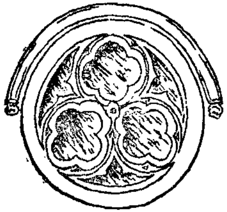
c. 1280

Fig. 2134.



c. 1300

Fig. 2135.



c. 1320

Fig 2136

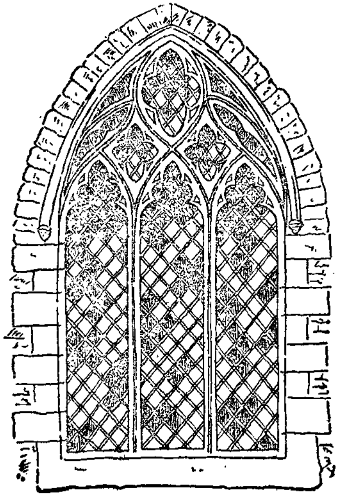
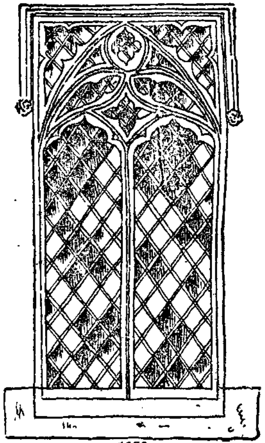


Fig. 2137.



c 1350

Fig. 2138.

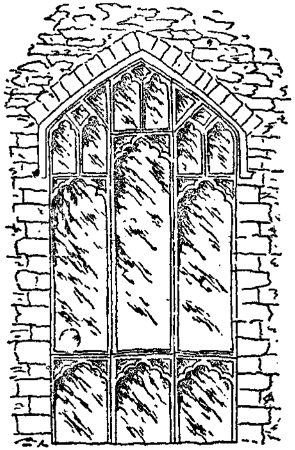
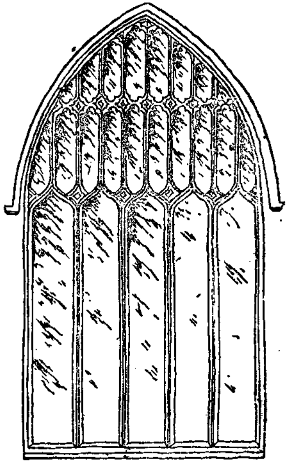


Fig. 2139.



c. 1500

Fig. 2140.

Figs. 2130 to 2132.—Plate Tracery. Figs. 2133 to 2140.—Bar Tracery.

beneath the arch and admitted more light. This elementary stage of ornamentation is called plate tracery. In early cusped circles there is a distinctive peculiarity in the cusping. In these the foils are produced without rising at all into the chamfer. This style has been called soffit cusping, because it rises directly from the soffit of the arch, and not, as subsequently, from the chamfer or slope of the arch side. Another marked peculiarity in early foils is that, instead of being segments of intersecting circles, they are formed from a series of distinct circles which all cut a larger circle within. Tracery in the cusping of which any of these peculiarities occur is invariably of an Early English when not actually of a Transitional character, as in the example (Fig 2134) from Meopham Church, Kent. The system shown in this example is known as bar tracery, in contradistinction to plate tracery, the patterns of the openings appearing to be formed by the intersection of various bars. This latter form arose naturally enough from the former by the multiplication of the piercings or apertures, till at last the plate disappeared, except such parts as were required to separate the openings and to connect the various parts.

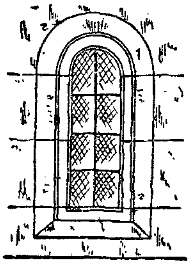


Fig. 2141.

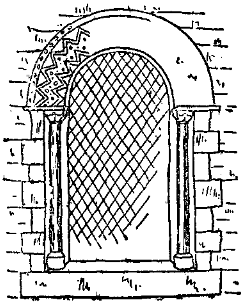


Fig. 2142.

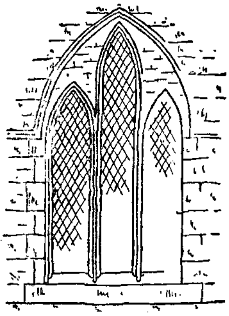


Fig. 2143.

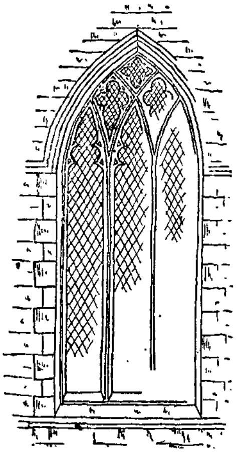


Fig. 2144.

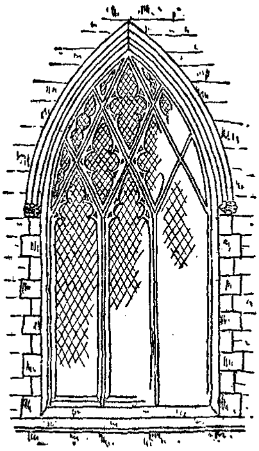


Fig. 2145.

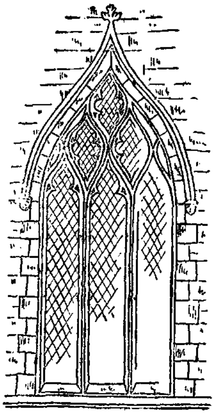


Fig. 2146.

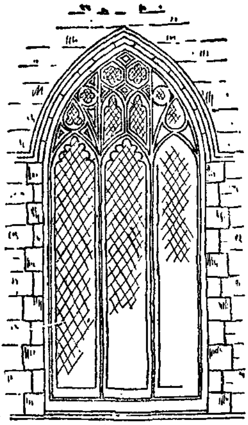


Fig. 2147.

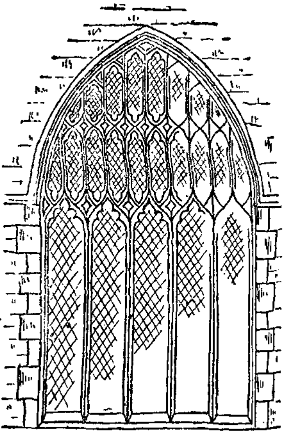


Fig. 2148.

Figs. 2141 to 2148.—Development of Window Tracery.

English Gothic Window Tracery.

In England, Gothic architecture proper began about the middle of the twelfth century, and was called Transitional, as pointed arches were used with details of Norman mouldings and enrichments; but Norman characteristics soon gave place to the more refined and slender Gothic features. The first definite type of

Gothic, as seen in the windows of the latter part of this century, took the form of long narrow openings, with pointed arched tops which very much resembled the lancet, hence the title lancet windows. Later, these windows were grouped, as in York Minster. In course of time these groups were enclosed under one arch, and the spandrils and head filled in with plate tracery—merely piercings to relieve the monotony of the flat surface. This tracery afterwards became moulded to resemble bent stone, and so obtained the name of bar tracery.

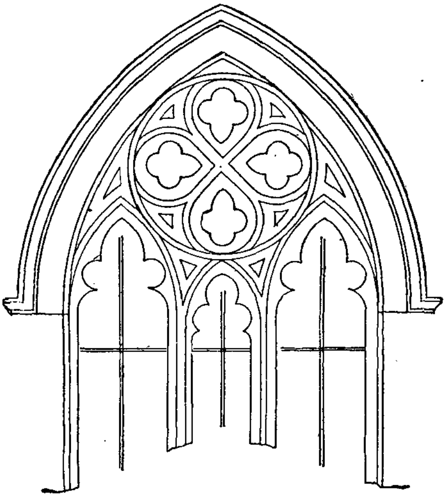


Fig. 2149.—Decorated Window.

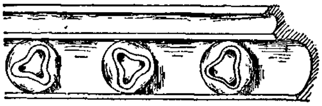


Fig. 2150.—Ball-flower Ornament.

The Early English style maintained its position for a long time, until the fourteenth century, when what is termed Decorated came into vogue, so called from the very elaborate arrangement which the traceries of the windows assumed, and the more ornamental character of the general details. This style lasted for about seventy years, and then a striking change took place. Straight lines were introduced into the traceries instead of graceful flowing lines, and the appearance assumed a severer aspect. The windows were lofty, divided by transoms, and the mullions ran through into the arch stones. The heads of the windows were flatter, and in some cases even square-headed windows are found. At the conclusion of this period, which dates about 1480, the Tudor or four-centred arches prevailed. The transoms of the windows were sometimes battlemented. As an example of Perpendicular (Tudor period), Henry VII.'s chapel at Westminster is typical. See Figs. 2141 to 2148 for a series of windows illustrating the development of tracery:—Fig. 2141, Handborough, Oxon., *c.* 1120; Fig. 2142, St. John, Devizes, *c.* 1160; Fig. 2143, Warmington, Northamptonshire, *c.* 1230; Fig. 2144, Dorchester, Oxfordshire, *c.* 1275; Fig. 2145, Stanton St. John, Oxon., *c.* 1320; Fig. 2146, Higham Ferrers, Northants, *c.* 1360; Fig. 2147, Merton College Chapel, Oxford, *c.* 1424; Fig. 2148, Swinbrook, Oxfordshire, *c.* 1500. Fig. 2149 shows a Decorated window in the earlier portion of the period, and Fig. 2150 the ball-flower ornament peculiar to the period. Fig. 2151 shows a Perpendicular window, and Fig. 2152 a running foliage ornament of the same period.

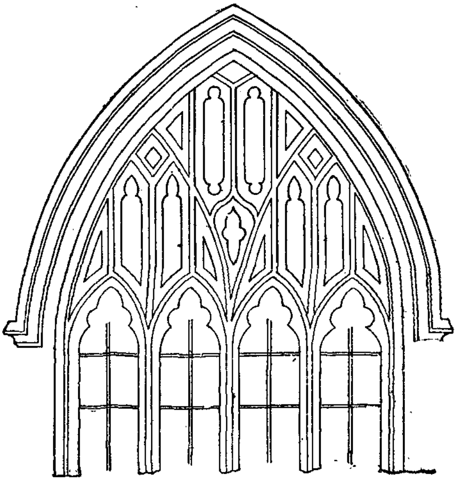


Fig. 2151.—Perpendicular Window.

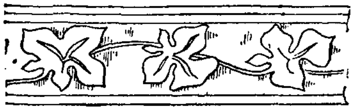


Fig. 2152.—Running Foliage Ornament

Geometrical Windows.

The Geometrical style, also known as Early

English, comprises mullioned windows with the upper part formed of an equilateral arch and filled with tracery composed chiefly of cusped circles, but invariably of rigid and symmetrical compass curves as in Figs. 2153 and 2154, both of which represent a window in Meopham Church, Kent, date about 1280. Fig. 2155 shows to a larger scale the cusping in the head. Following the tendency of the time, the windows became higher, and were filled with more delicate tracery, having ogee or flowing curves and being surrounded by more delicate mouldings, as in Fig. 2156. In the extreme Decorated style, the tracery became flame-like or flamboyant when the limit of ornamentation was arrived at. "In the triforium arcades of buildings erected in the Romanesque style of architecture (from 1000 to 1145 A.D.), two arched openings are often found, having a large semicircular arch enclosing them. The tympanum or spandril between them is invariably enriched by sinkings or piercings of various kinds. As the time went on, windows were treated in a similar manner.

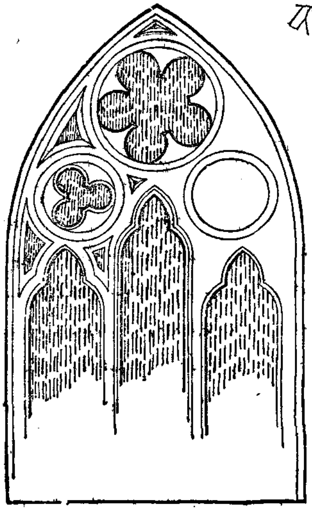


Fig. 2153.

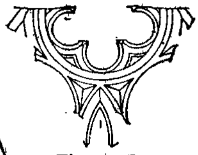


Fig. 2155.

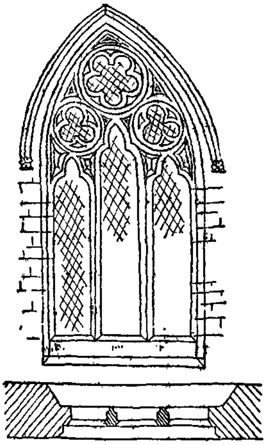


Fig. 2154.

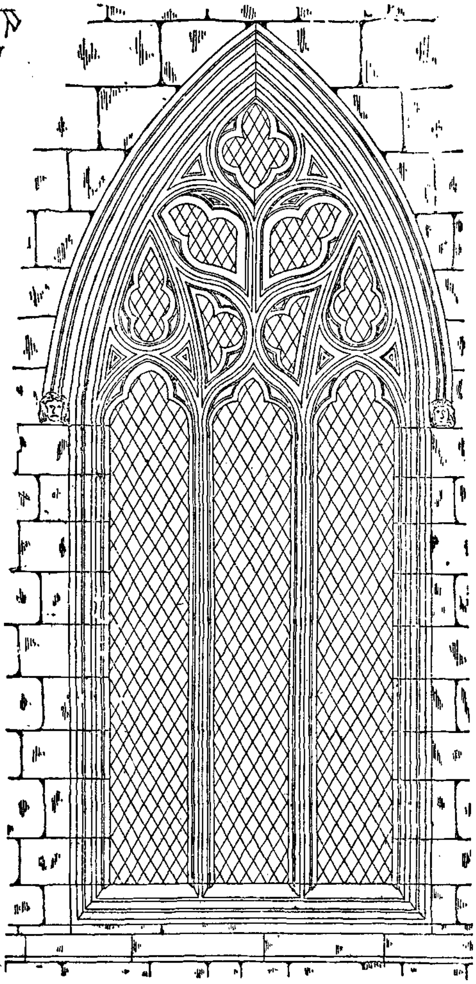


Fig. 2156.

Figs. 2153 to 2155.—Three-light Window in Geometrical Style. Fig. 2156.—Three-light Window in Decorated Style.

and it is then that we have the origin of tracery; such perforations are known as plate tracery, this being developed to a great extent during the earlier part of the Early English periods (1189 to 1220 A.D. being the approximate dates). In the latter part of this style the smaller spandrils of the space between the arches were filled in with geometrical designs, and the perforations followed the line of the various arches and gave to the stonework the appearance of having been bent into shape. This is the earliest form of bar tracery, and was mainly composed of circles, or parts of circles, intersecting one another. The windows belonging to this period are divided into a series of lights of compartments by mullions, often with columns attached. The most intricate design that was adopted could be easily drawn with the aid of compasses, the best result being therefore somewhat stiff and

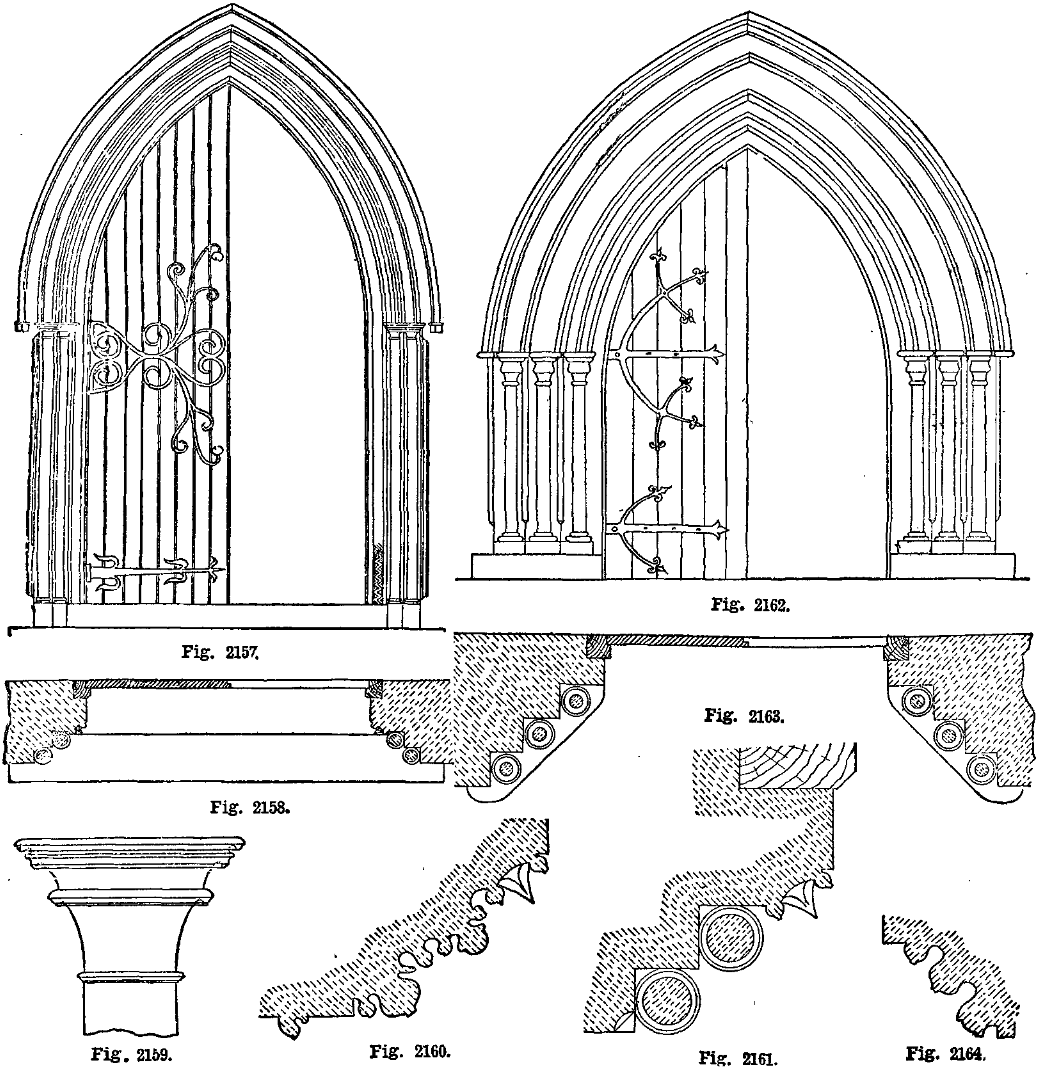


Fig. 2157.
Fig. 2158.
Fig. 2159.
Fig. 2160.
Fig. 2161.
Fig. 2162.
Fig. 2163.
Fig. 2164.

Figs. 2157 to 2164.—English Gothic Doorways and Details.

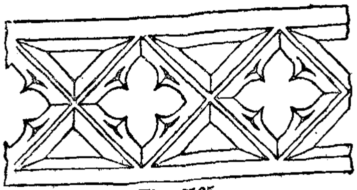


Fig. 2165.

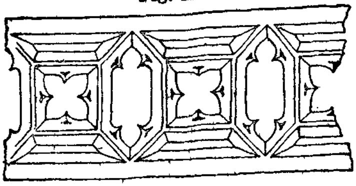


Fig. 2166.

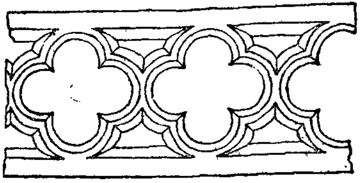


Fig. 2167.

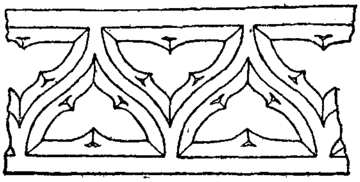


Fig. 2168.

Figs. 2165 to 2168.—Gothic Perforated Parapets.

formal, but for the beautifying effect of the quatrefoils and cinquefoils within the geometrical figures."

Decorated or Curvilinear Windows.

"In the Decorated or Curvilinear style," our authority continues, "which was the outcome of the previous one, the geometrical figures were not strictly adhered to; flowing lines were adopted, thus giving unlimited scope to the designer. The heads of the windows were almost always equilateral arched and filled with a maze of tracery flowing gracefully, there being no abrupt angles or intersections." Cusping was freely used, every spandril being ornamented with two or more cusps. Some of the tracery took the form of leaves springing from a main branch, others were reticulated (having the appearance of network); while in France, at a rather later date, the tracery assumed a flame-like appearance known as Flamboyant. The Decorated style was undoubtedly the perfect Gothic. It displays a freedom of treatment positively unequalled in any other style of the Gothic; a spirit of unrest seems to pervade the design, it being no doubt a type of the spirit of the age in which it prevailed. It is impossible to imagine a greater contrast than that which is represented by comparing this style with that of the Grecian Doric. They are certainly the two extremes of architecture.

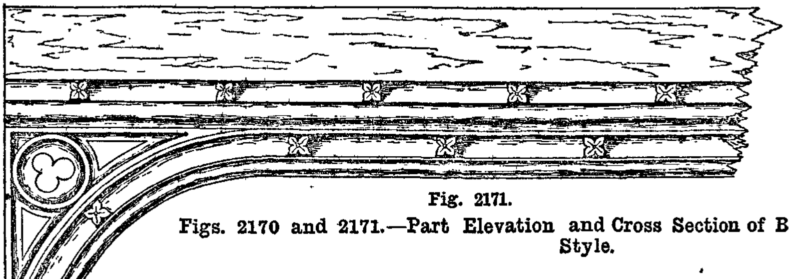


Fig. 2171.

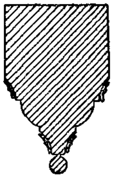


Fig. 2170.

Figs. 2170 and 2171.—Part Elevation and Cross Section of Beam in Perpendicular Style.

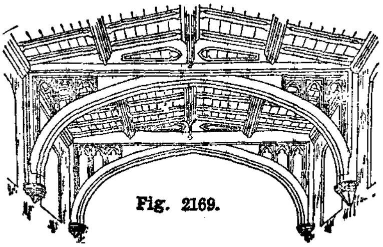


Fig. 2169.

Fig. 2169.—Roof over Church Nave.

English Gothic Doorway.

Figs. 2157 to 2161 show elevation, cross-section, and details of a doorway, 6 ft. wide, of the Early English Period of Gothic, with shafted and moulded jambs, and moulded arch, in stonework. The oak doors and frame are also shown. Alternative sketches are presented by Figs. 2162 to 2164.

Gothic Perforated Parapets.

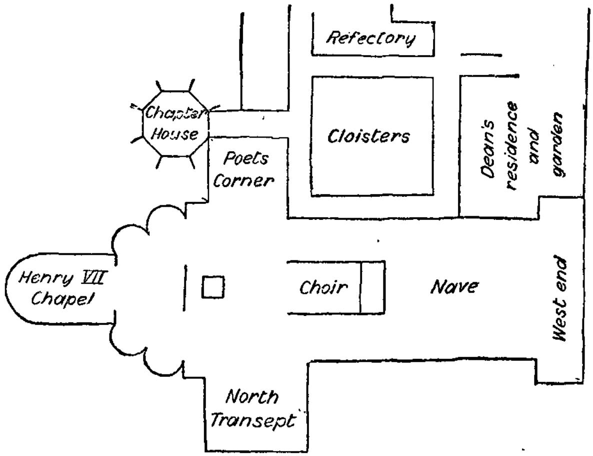
Figs. 2165 to 2168, which are adapted from R. S. Burn's "Architectural Drawing Book," show Gothic perforated parapets.

Examples of the Perpendicular Style.

Fig. 2169 shows the roof over the nave of Ridlington Church, Oxon., date about 1450. Fig. 2170 represents the part elevation, and Fig. 2171 the section, of a beam, moulded and enriched, belonging to the Perpendicular style.

Architectural Styles in Westminster Abbey.

Regarding the positions of the different styles of architecture in Westminster Abbey, Saxon and Early Norman are found in the refectory and parts adjoining; Early English (1216 to 1272), the major part of the chapter-house, part of the cloisters, and the east end of the Abbey, including north and south transepts; later Early English (1272 to 1307), the choir and adjoining parts of nave and cloisters; Early Perpendicular, but in thirteenth-century style (1362 to 1500), the west end of nave and west front of Abbey; Late Perpendicular, Henry VII. Chapel. The western towers, by Sir C. Wren and Hawksmoor, 1772 to 1740. North transept refaced by Sir Gilbert Scott, 1880 to 1892. Fig. 2172 shows a sketch-plan of Westminster Abbey, indicating the various parts referred to above. Between Henry VII.'s Chapel and the choir is Edward the Confessor's Chapel.



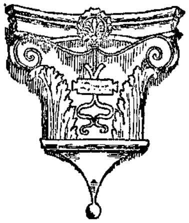


Fig. 2173.

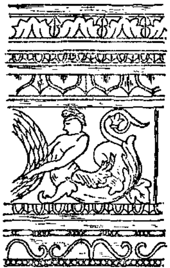


Fig. 2174.

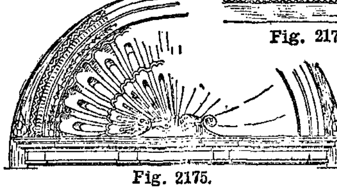


Fig. 2175.

Fig. 2172.—Sketch Plan of Westminster Abbey. Fig. 2173.—Tailpiece. Fig. 2174.—Part of Frieze. Fig. 2175.—Italian Renaissance Shell Ornament.

Renaissance Style.

The Renaissance style of architecture was the result of the revival of classic literature at the beginning of the fifteenth century. It originated in Italy, and its degeneration had already set in before it spread to England at the beginning of the sixteenth century. It came then through the Dutch, and formed a transitional style known as Elizabethan; but Inigo Jones, early in the seventeenth century, introduced a purer style, founded upon the best Italian models, and this was continued by Wren, although with greater freedom of treatment. It continued in vogue until about 1850, when a Gothic revival took place; which, again, gave way towards the close of the century in favour of a free Renaissance treatment. The leading features of Elizabethan Renaissance distinguishing it from English Mediæval are: Elevations symmetrical on either side of a central line. Broad terraces with balustrades. Windows square, very large and high, divided by mullions and transoms, and placed in long rows to form the leading feature of each storey. Oriel windows. Gables with scroll outline. Balustered skyline above a main cornice. Pierced parapets absent. Roofs high-pitched or low-pitched, frequently domed. Ogee-topped lanterns. Ornamental plastered ceilings, coffered, barrel, or flat. Entrances with pediments and pilasters. Ball finials numerous. Arches semicircular or elliptical. Ionic and Corinthian capitals to columns and pilasters. Sculpture used as an accessory.

Renaissance Mouldings and Ornament.

Fig. 2173 is a sketch of a tailpiece from a doorway of Santa Maria, Venice, A.D. 1632; Fig. 2174, a frieze from Lodi, Italy; Fig. 2175, an example of the Italian Renaissance shell ornament; and Fig. 2176, an example of French Renaissance, being a capital from the House of Francis I. at Orleans, A.D. 1540.

Elizabethan Architecture.

Elizabethan (1558-1603) is a transition style between the Tudor or Late Gothic and the revived Classic architecture, and was transmitted through the Dutch. It was applied chiefly to large country houses or mansions, and to some of the colleges at Oxford and Cambridge. It is not a church style. Its chief characteristics of planning are: (1) Symmetrical arrangement on each side of a central line. (2) The great hall, a survival of the ancient style of domestic building from the time when the hall was the only apartment. The importance of the hall was gradually diminished by the addition of more and more private rooms for the family, and of offices, etc. The walls were lined to a height of 8 ft. or 10 ft. with oak panelling, and trophies of the chase, tapestry, heirlooms, relics, etc., were arranged on the space above. (3) The broad staircase of oak, with heavily carved newels, pierced balustrading, and rich carving. (4) The great gallery on the first floor, extending the whole length of the house, relieved by large projecting bays, containing windows in oak to their full height, and the ceilings richly modelled in plaster. Its chief characteristics of detail were: Gables scroll-shaped and terminated with ball finials; balustered parapet above the main cornice; roofs fairly high-pitched; smaller parts domed or ogee-shaped; windows very large, mostly square, with mullions and transoms, and filled with lead-lights containing coats of arms; walls of the principal room panelled in oak to their full height, and the ceilings richly modelled in plaster.

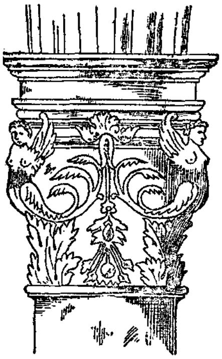


Fig. 2176.—Capital, French Renaissance.

Jacobean Architecture.

Jacobean architecture had its period between the years 1603 and 1625. There was no break in style between Elizabethan and Jacobean, but a gradual transition from modified Gothic to a more pronounced Italian style, and an increasing preference for stone over brickwork. Many Jacobean farmhouses were built of stone, with square-headed windows, stepped or scroll gables, and tiled roofs, being a great advance on previous farm residences. They are even now to be seen dotted about the country, and adding picturesque charm by their lichen-covered exteriors.

Queen Anne Architecture.

The Queen Anne style of architecture, following the Elizabethan, came to England through the Dutch, being introduced by King William III. at the beginning of the eighteenth century, and developed during the reign of Queen Anne. It was based on the Gothic styles which had preceded, but was largely imbued with the Classic Renaissance, particularly in some of its details. They are mostly solid rectangular buildings in plan. The usual material for the walls is red brickwork, with pilasters, cornices, panels of carved ornamentation, and other decorative features, such as aprons under the window-sills, executed in cut bricks, showing many fine examples of workmanship. For arches, niches, and window heads, very finely jointed brickwork was used. The wall was generally finished by a crowning cornice of considerable projection under a high-pitched roof with hipped ends. The roofs were steep and of considerable area in one plane, sometimes with prominent picturesque dormers, high brick chimneys, and lofty gables. The brickwork of the gable-end was sometimes broken up into steps, ramps, curves, and scrolls, terminating with ball finials at the eaves, as in Elizabethan work, surmounted by a pediment at the apex. The joiners' work is an important feature of the style; it is heavy and massive, and lends considerable charm to the structure. The windows are rectangular, and arranged with sashes, the sash bars being very thick, and the whole space divided into small square panels. The frames and bars are usually painted white, but sometimes a light or dark sage green; the white, however, being the most effective. The doors are heavily moulded, and are frequently surmounted by pediments carried on carved brackets or pilasters. The woodwork of the staircases is very bold and effective, the newel posts and balusters being massive, and ornamented chiefly with turned mouldings, the handrail large and the staircase wide, to match the proportions of the other parts. It is a very popular style among architects of the present day, principally from the impetus given by the designs of Norman Shaw, Ernest George and Peto, and Robson of Board School celebrity.

Scottish Baronial Style of Architecture.

The Scottish Baronial style of architecture resembles that of the Renaissance châteaux of France, and the Scottish noblemen seem even to have employed French architects. The style may be said to have been in vogue from the beginning of the sixteenth to the end of the seventeenth century. Corner turrets corbelled out from the upper floors were very characteristic of the style. In this, as in all Scottish architecture, the gables had stepped outlines. This was originally from rude workmanship, but was afterwards a distinctive feature called "crow stones." There were as a rule few windows near the ground, the upper rooms being used as the chief apartments, but these castles were not built as fortresses. Although the battlements were retained, it was only as ornament, and the corner turrets were generally useless for defending the walls, as the window was situated at the external angle. The absence of openings near the base gave a pleasing effect of solidity and strength, "which is characteristic of the people and their national disposition." In some examples, the turrets are covered with high-pitched conical roofs; in others, the roofs were kept out of sight and the tops castellated. The details are as a rule Classic, but are treated with considerable freedom. Newark Castle, near Selkirk, is a fairly good example of the style. Another description: The castles and semi-fortified houses of Scotland form a group apart, possessing strongly marked and well-defined character. They are designed in a mixed style, in which the Gothic elements predominated over the Classic ones; but the Scottish domestic Gothic, from which the new style was partly derived, had borne little or no resemblance to the florid Tudor of England. It was the severe and simple architecture of strongholds built with stubborn materials, and on rocky sites where there was little inducement to indulge in decoration. Dunstaffnage Castle and Kilchiern Castle may be referred to as examples of these plain, gloomy keeps, with their stepped gables, small loops for windows, and sometimes angle turrets. The Classic elements of this style were not drawn direct from Italy, but came from France. The Scots, during their long struggles with the English, became intimately allied with the French, and it is not surprising that Scottish Baronial architecture should resemble the Early Renaissance of French châteaux very closely. The hardness of the stone in which the Scottish masons wrought forbade their attempting the delicate detail of the François I. ornament; and the difference in the two climates justified in Scotland a boldness which would have appeared exaggerated and extreme in France. Many castles were erected in the sixteenth and following centuries in Scotland, or were enlarged or altered. The most characteristic features in almost all of them are short, round angle turrets, thrown out upon bold corbellings near the upper part of towers and other square masses. These are often capped by pointed roofs; and the corbels that carry them, which are always of a bold, vigorous character, are frequently enriched with a kind of cable ornament that is very distinctive. Towers of circular plan, like bastions, and projecting from the general line of the walls or at the angles, constantly occur. They are frequently crowned by conical roofs, or finished with gables; or otherwise, if made square near the top, by a series of corbels. Parapets are in general use, and are almost always battlemented. Roofs when visible are of steep pitch, and their gables are almost always of stepped outline; while dormer windows, frequently of fantastic outline, are not infrequent. Chimneys are prominent and lofty; windows are square-headed and, as a rule, small, sometimes retaining the Gothic mullions and transom, but in many cases these features are absent. Doorways are generally arched, and not often highly ornamented. Cawdor Castle, Glamis Castle, Fyvie Castle, Castle Fraser, the old portions of Dunrobin Castle, Tyninghame House, the extremely picturesque palace at Falkland, and a considerable part of Stirling Castle, may be all quoted as examples of this thoroughly national style.

Adam Architecture in London.

Robert Adam, in 1757, with a view to introducing a purer Classic architecture into London visited Rome and other Italian cities, noting particularly the arrangement and ornamentation of the Baths of Titus and Diocletian at Rome and the Palace of Diocletian in Dalmatia, facing the Bay of Spalatro. Upon his return he, assisted by his three brothers, carried out all the principal residential architecture in London during the latter half of the eighteenth century, their father having been similarly engaged during the first half of the century. Edinburgh, Glasgow, Newcastle, Dublin, Bath, and other important places, have also many specimens of Adam architecture. In London are the Adelphi (including the home of the Society of Arts), Portland Place, Finsbury Square, Great George Street (Westminster), Spring Gardens, Bedford Square, Bryanston Square, Berkeley Square, and Terraces at Kensington, Walworth, Old Kent Road, Kennington, etc., Montague House (Portman Square), Kenwood House, (Hampstead), Osterly House (Brentford), Sion House (Isleworth), etc., etc., all the work of the brothers Adam, of whom Robert was the leading spirit. The chief features of the Adam architecture are Classic feeling, with great simplicity of character, and accurate proportions. Columns engaged with the front wall, or pilasters, frequently in cement. A triple window in the centre, the middle light being arched over. Double doors at entrance, with wide lunette fanlight, say one-third of a circle. Garlands and festoons abundantly used in decoration, also oval medallions. Houses with curved fronts. Ceilings, chimney-pieces, doors, and even furniture, all tastefully designed to suit the style of architecture. A typical building is Lansdowne House, Berkeley Square.

Origin and Characteristics of the Adam Style.

The following further account of Adam and his work will be useful as representing a model answer to a question set in an architectural examination:—During the latter half of the eighteenth century, the four brothers Adam to all practical purposes had the monopoly of the chief architectural undertakings in the United Kingdom. Their father had been, during the early part of the same century, a noted architect with a practice especially extensive in Scotland, where the French Renaissance style, emboldened to suit the more rigorous climate, was in vogue. Robert Adam, one of the brothers, could thus at an early age have commenced business with a wide connection; but before doing this, he was determined to study architecture abroad, and in 1757 he went to Italy with the object of learning a Classic style which would be suitable to domestic architecture at home. He failed to do this in Italy; but, sailing across to the Bay of Spalatro, in Dalmatia, he found all he wanted in the immense palace built by the Emperor Diocletian after his abdication. The Adam style was strictly Classical and refined, being a great contrast to all existing architecture of the period, which was very heavy and florid. Robert had a keen sense of proportion, which he displayed in his use of the column, garland, or doorway. His idea was that the façade of a building should have an expression of unity, harmony, and even motion, and it was for this reason that, having in the main only common stock bricks of which he could build his houses in London, he rejected stone for his dressings, saying that it made the frame look too heavy for the picture, and adopted in its stead a cement invented by a Swiss clergyman named Liardot, which, whilst embodying the endurance of stone, also possessed the flexibility of stucco. This cement Adam used for both outside and inside work; externally for pilasters, architraves, cornices, and mouldings of all kinds, and inside chiefly for his beautifully designed ceilings; and it is worthy of note that there are specimens of his work executed in this material more than a hundred years ago, which still retain their pristine beauty and vigour of outline. Perhaps the most remarkable feature in the façade of an "Adam house" is the system of fenestration. He arranged his windows as a rule so as to indicate to an observer from the outside something of the internal arrangement of the house. A good example of this may be seen at Boodle's Club, in St. James's Street, where there is a porch on each side of the first storey, and it is obvious that the central room on the first floor is the chief and largest assembly room for the members. As it is the largest window in the house, admitting much light, it is the only one treated with any amount of decoration; and this decoration, and the style of the window, are also typical of "Adam architecture." It is one window divided into three by two columns, and covered by a cornice that spans the central bay by a semicircular arch; and this arch is filled in with an escalop-shell device. This design he obtained from Spalatro, and he used it somewhere or other in nearly every house he built; thus at Boodle's Club it is the chief window to the chief room; at a house in Conduit Street it adorns a garret window; and at Stratford Place it forms the landing window to the back entrance. Adam built many rows of houses, terraces, places, and squares in London, such as Fitzroy Square, St. James's Square, Berkeley Square, The Adelphi, Stratford Place, and Portland Place; and it was his idea that, since a number of adjoining houses really form one building, therefore the architecture of the whole façade should be treated as if it were one house, and this idea has been carried out in Stratford Place, where the Classic tetrastyle is the order used. The three central houses are distinguished by four flat engaged columns, crowned by a graceful pediment. The cornice is continued each way till the three end houses are reached, where pediment and tetrastyle again occur, although they are not quite so marked. Adam was always fond of varying the shape of his rooms; hence he not only had them square and oblong, but also round, oval, and octagonal, and he also often made his ceilings circular. In fact, he used the arch wherever he possibly could consistently with the proper expression he wanted to obtain. In one of his houses he made the arches to the first storey thick and heavy, in the next flatter and thinner, and to the top storey, which had least to carry, he put flat lintels, thus conveying to the spectator the sensation of strength, buoyancy, and motion. Adam never put in unnecessary decoration, but tried to make each separate detail or ornamentation give one homogeneous expression to the whole; he often designed a house throughout—not only the external and internal decorations, but even the wallpaper, furniture and carpets, and throughout the whole he maintained one graceful uniformity and expression, which truly made anything of his a real thing of beauty.

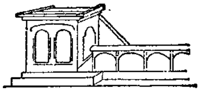


Fig. 2177.—Ambo.

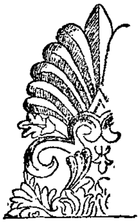


Fig. 2178.—Antefixa.

Some Terms in Architecture and Ornament Defined and Illustrated.

Ambo.—Ambo (Fig. 2177) was a kind of pulpit, rostrum, or reading-desk, placed near the west end of the choir of early Christian churches, from which the Gospels and Epistles were read, but sometimes there were two ambones, one for the Gospel and the other for the Epistle. The typical ambo had two ascents to it by steps, one on the east side and the other on the west. In the upper part were usually two steps, from the higher of which the Gospel was read, and from the lower the Epistle. It is now seldom seen except in Eastern churches. Examples still exist, however, in the churches of San Lorenzo, St. Laurentius or St. Laurence; San Clemente or St. Clement, and St. Pancratius or St. Pancras, all at Rome; in the two former churches they are covered with mosaics. According to

Ciampini, the ambo fell into disuse about the beginning of the fourteenth century.

Antefixæ.—Antefixæ (Fig. 2178) are the upright ornamental blocks placed at the top of the cornice and against the eaves at intervals along the side of a roof to hide the overlapping edges of the roof tiles. They were frequently made of terra cotta, with honeysuckle or other decoration moulded on them. The name is also applied to the lions' or other heads carved on the upper part of the cornice, either for ornament or to serve as outlets or spouts to carry off the water, as on the Temple of the Winds at Athens. Fig. 2178 shows an antefixa from the Propylæum at Athens.

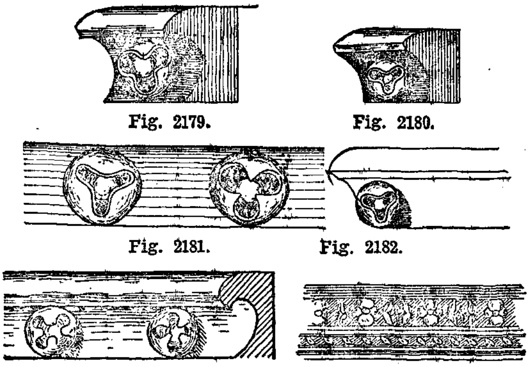


Fig. 2179. Fig. 2180.

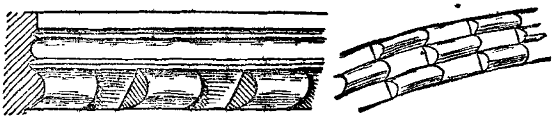
Fig. 2181. Fig. 2182.

Fig. 2183. Fig. 2184.

Figs. 2179 to 2184.—Ball-flower Ornaments.

Ball-flower. — Varieties of the ball-flower ornament, which belongs to the Decorated period, are shown by Figs. 2179 to 2184.

Billet.—Billet is an ornament used in mouldings of string-courses and archivolts of openings in the Norman period of architecture. It consists of short small cylindrical or semi-cylindrical pieces 2 in. or 3 in. long, placed in a hollow moulding at intervals equal to about their own length. Sometimes one, two or three rows of billets are applied. Figs. 2185 and 2186 show examples.



Figs. 2185 and 2186.—Billet.

Boss.—Examples are shown by Figs. 2187 to 2191; it is employed at the intersections of ribs or mouldings, and at the terminations of drip-stones, etc.

Brattishing. — Brattishing, brandishing, or brattice work is the term applied to a perforated or embattled parapet as Fig. 2192, or the crest of open carved work on the top of a shrine.

Broach Spire.—Broach spire is a spire rising direct from a square tower without having an intervening parapet. The spire may be square or octagonal; when it is square there is generally a bell-cast given to the eaves, as shown in Fig. 2193; but when the spire is octagonal the same effect is obtained by short angular pieces on the corners of the tower, as shown in Fig. 2194. Sometimes pinnacles are carried on these angles, which are then called broach pinnacles, and the angles themselves are often called broaches.

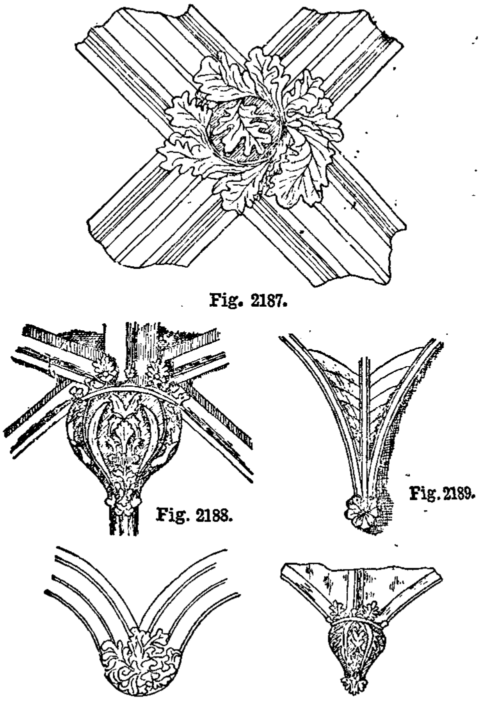


Fig. 2187.

Fig. 2188.

Fig. 2189.

Fig. 2190.

Fig. 2191.

Figs. 2187 to 2191.—Bosses.

Cable.—This is an ornament applied, in the Norman style, to a convex or rounded moulding; it represents, as shown in Fig. 2195, the twisted strands of a rope or cable,

Chapter-house.—This is the apartment in connection with a cathedral or collegiate church in which the dean and canons meet to transact business. It usually leads out of the east side of the cloisters, and is generally polygonal in plan, with a vaulted roof.

Coffers.—Coffers are the square depressions or sunk panels in vaulted roofs and domes and flat ceilings, often ornamented in the centre.

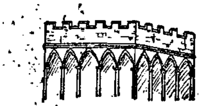
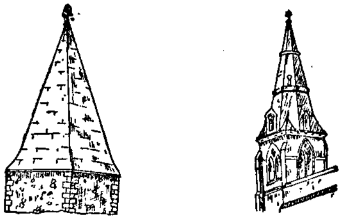


Fig. 2192.—Brattishing.

Columns: Their Spacing.—Fig. 2196 shows the spacing of columns in Classic architecture. Systyle refers to the spacing of the columns in Classic architecture, called also close columniation, the columns being only two diameters apart. The closest is called Pycnostylar, where the columns are only 1½ diameters apart; Eustyle, or medium spacing, 2¼ to 2½ diameters apart (authorities vary on this point); Diastyle, or wide spacing, 3 diameters apart; and Arœstyle, or very wide spacing, 3½ diameters apart.



Figs. 2193 and 2194.—Broach Spires.

Columns: Their Arrangement.—Peripteral is applied to temples having columns or a colonnade all round the cella or central block, the front and back being frequently in double rows (Fig. 2197). According to another account, a peripteral temple consisted of a circular building with a portico of six columns on each front and a colonnade of eleven columns on each side of the building, the columns at the angles being included in each computation. Pseudo-peripteral is the term applied to temples in which the columns on the sides are attached to the walls, having a portico only in front, as in the Maison Carrée at Nismes and the Temple of Fortuna Virilis at Rome. The pseudo-peripteral was like the peripteral in form, but with the breadth of the building increased so that it became united with the columns on each side. The term "peripteral octastyle" refers to the general arrangement of columns in a temple; the first half of the term signifying columns all round, and the second half that there are eight columns in line at the ends, as in the Parthenon at Athens (Fig. 2198).

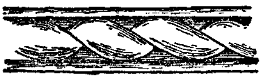


Fig. 2195.—Cable Ornament.

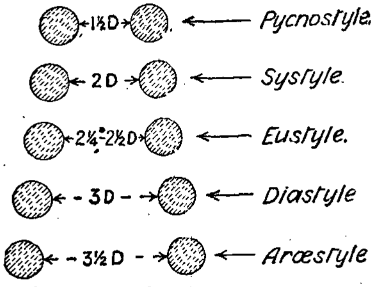


Fig. 2196.—Spacing of Columns.

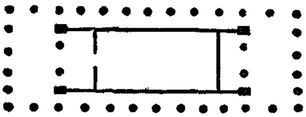


Fig. 2197.—Peripteral Arrangement of Columns.

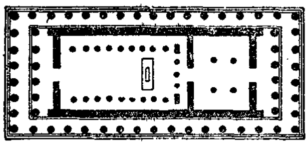
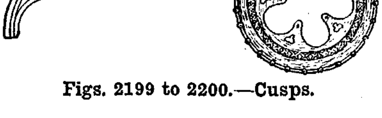


Fig. 2198.—Peripteral Octastyle Arrangement of Columns.



Figs. 2199 to 2200.—Cusps.

Corona.—Corona is the flat square projecting member of the cornice below the cymatium and above the frieze in Grecian and Roman architecture, bevelled on the under side to throw off the water from the roof clear of the ornaments and mouldings below. Called also the drip or larmier.

√ **Cortile.**—Cortile is adopted from the Italian, and signifies an internal area or courtyard surrounded by an arcade.

√ **Cusp.**—Cusp (Figs. 2199 and 2200) is the name given to a curved projection in the soffit of a moulding, formed by the termination of circular arcs, whether in a sharp or blunt point; as, for instance, the junctions of the foils in trefoil, quatrefoil, and other tracery.

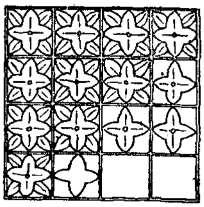


Fig. 2201.

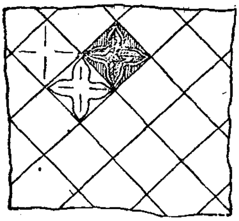


Fig. 2202.

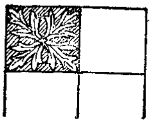


Fig. 2203.

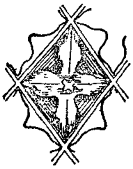


Fig. 2204.

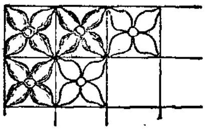


Fig. 2205.

Figs. 2201 to 2205.—Diaper Patterns.

Diaper.—Varieties of the diaper pattern are presented by Figs. 2201 to 2205. The pattern is disposed regularly in squares or lozenges.

Die.—Die is a naked square cube. The plain flat surface, or body, of a pedestal, between its base and cap, is called the die of the pedestal. The dado of a wall is similar in position and character.

√ **Dipteral.**—Dipteral is the term used to signify a temple with a double row of columns on each of its flanks or sides. It was one of the seven orders of sacred buildings, and had an octastyle in front and rear.

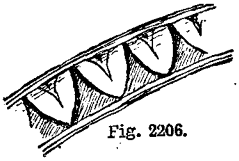


Fig. 2206.

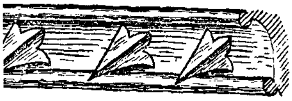


Fig. 2207.

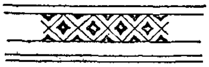


Fig. 2208.

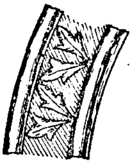


Fig. 2210.

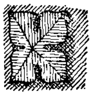


Fig. 2211.

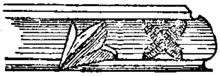


Fig. 2209.

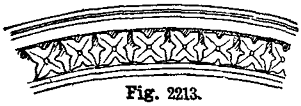


Fig. 2213.

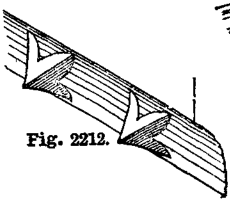


Fig. 2212.

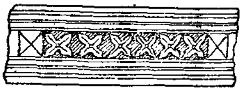


Fig. 2214.

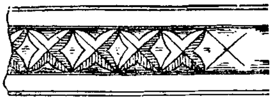


Fig. 2215.

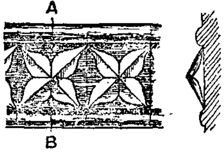


Fig. 2216.

Figs. 2206 to 2216.—Dog-tooth Ornaments.

Dog-tooth Ornament.—Typical examples of dog-tooth ornament are illustrated by Figs. 2206 to 2216. It gets its name from the fact that it is an adaptation from the dog-tooth violet. It prevailed during the thirteenth century in the Early English style of architecture. Fig. 2217, from New Shoreham church, Sussex, and Fig. 2218, from St. Ethelred's, Norwich, show Early English ornament allied to the zigzag and dog-tooth.

Façade.—Façade is the term used for the principal elevation of a public edifice, as the

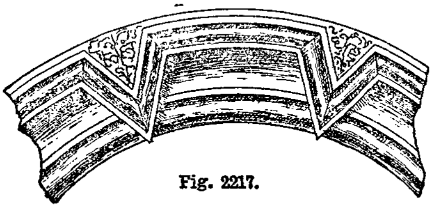


Fig. 2217.

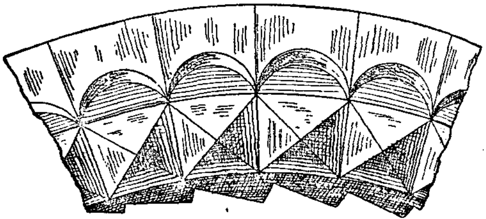


Fig. 2218.

Figs. 2217 and 2218.—Early English Ornament allied to Zigzag and Dog-tooth.

west front of a cathedral. See Fig. 2219, which shows the façade of the cathedral at Orvieto.

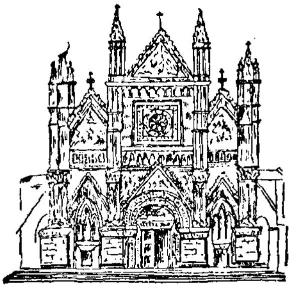


Fig. 2219.—Façade of Orvieto Cathedral.

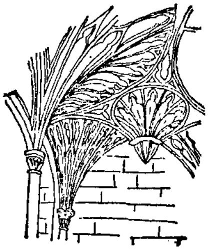


Fig. 2220.—Fan Vaulting.

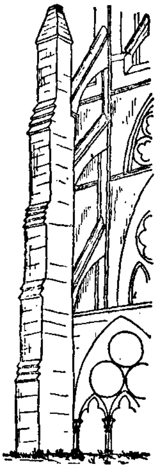


Fig. 2221.—Flying Buttress.

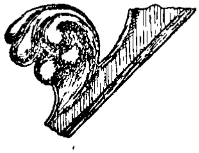
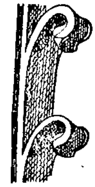
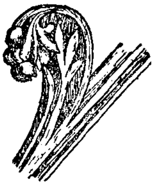
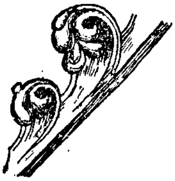
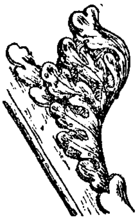


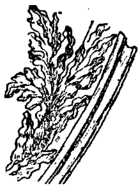
Fig. 2222.

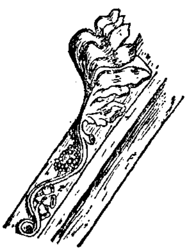

Choir, Lincoln Cath., c. 1200.
Fig. 2223.

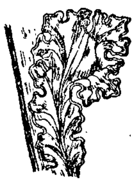

Salisbury Cath., c. 1240.
Fig. 2224.

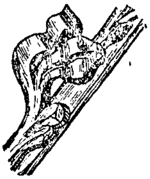

York Cathedral, A.D. 1255.
Fig. 2225.

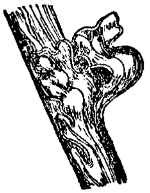

Southwell Minster, Notts., c. 1300.
Fig. 2226.

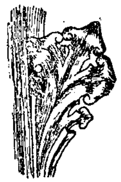

Hawton, Notts. c. 1320.
Fig. 2227

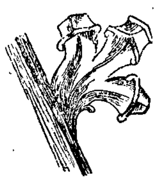

St. Mary's, Beverley, c. 1390.
Fig. 2228.


Lincoln Cath., c. 1330.
Fig. 2229.


Crick, Northamptonshire, c. 1420.
Fig. 2230.


St. Alban's Abbey, c. 1420.
Fig. 2231.


Litcham, Norfolk, c. 1450.
Fig. 2232.


Solihull, Warwick.
Fig. 2233.

Figs. 2222 to 2225.—Foliage of Early English Period. Figs. 2226 to 2229.—Foliage of Decorated Period. Figs. 2230 to 2233.—Foliage of Perpendicular Period.

Fan Vaulting.—Fan vaulting (Fig. 2220) is the term applied to the peculiar ornamented vaulting of the Perpendicular period. It consists usually of a number of inverted parabolic or other semi-conoids of hollow curved outline, springing from slender side shafts or wall ribs on corbels, and branching like a fan, divided by pointed arches, and repeated in similar conoid pendants over the intervening soffit of the vault. The whole of the surfaces is covered with tracery, and the intermediate flat parts are cusped or ornamented with bosses.

Flèche.—Flèche is a small wooden spire or turret surmounting a roof in Gothic work.

Flying Buttress.—Fig. 2221 shows a flying buttress at Westminster Abbey.

Foliage.—The foliage of the Early English style consists chiefly of trefoil leaves, which are varied in a great number of ways. The foliage round the capitals consists of these leaves supported on stiff stalks rising from the necking, and is named from this "stiff-leaf foliage." This is a distinctive mark of Early English work. (See the four varieties of crockets shown in Figs. 2222 to 2225.) The foliage of the Decorated period is more faithfully copied from nature than in the other styles. The surface of the walls was sometimes covered with flat foliage arranged in squares called diaper-work, and is believed to have originated in an imitation of tapestry wall-hangings. (See Figs. 2226 to 2229 for crockets.) The foliage of the Perpendicular style is also copied from nature to a certain extent, and although very beautifully executed there is a squareness about the leaves which rather detracts from the general beauty. The foliage of the capitals often resembles a wreath of flowers twisted round the top of the pillar. (See Figs. 2230 to 2233 for crockets.)

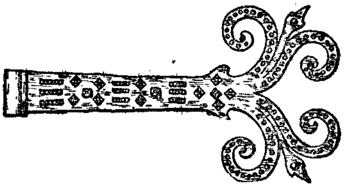


Fig. 2234.—Foliated Hinge.

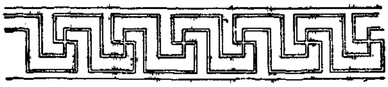


Fig. 2235.

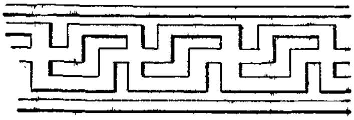


Fig. 2236.

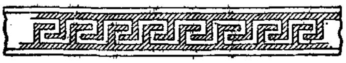


Fig. 2237.

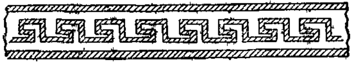


Fig. 2238.

Figs. 2235 to 2238.—Fret Ornament.

Foliated Hinge.—An example of a foliated hinge is illustrated by Fig. 2234.

Forum.—Forum was the open public space in Rome round which were situated the law courts and public buildings. It was here that the citizens transacted their business, and where orations were delivered to the people. The modern equivalent is the "Place" in Continental towns, and the market-place in provincial towns in England. Trafalgar Square in recent times has been used in the same manner as the old Roman forum, but only by "permission."

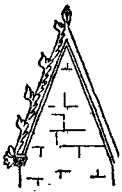


Fig. 2239.

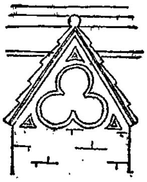


Fig. 2240.

Figs. 2239 and 2240.—Gablets.

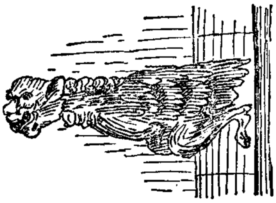


Fig. 2241.

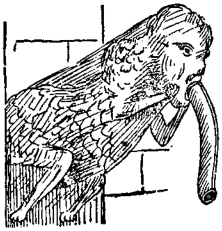


Fig. 2242.

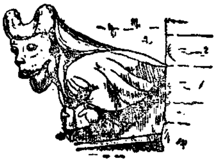


Fig. 2243.

Figs. 2241 to 2243.—Gargoyles.

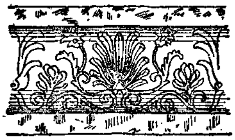


Fig. 2244.

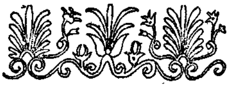


Fig. 2245.

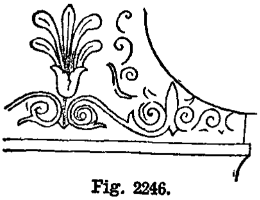


Fig. 2246.

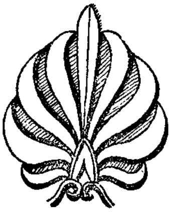


Fig. 2247.

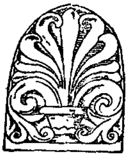


Fig. 2248.

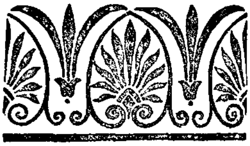


Fig. 2249.

Figs. 2244 to 2249.—Honeysuckle Ornament.

Fret.—Examples of the well-known fret are shown by Figs. 2235 to 2238.

Gablet.—A gablet (Figs. 2239 and 2240) is the termination of a buttress in Gothic architecture, in the form of a gable-shaped finial.

Galilee.—This is a porch or chapel at the entrance of a church, usually found in Gothic buildings, and particularly in abbey churches. At Lincoln Cathedral it is a porch on the west side of the south transept. Sometimes it is the lower part of the tower formed into an open porch with a doorway into the church; at other times it is the west end of the nave when separated from the remainder by a step or other division. Alessandro designed the façade of the church of San Giovanni Laterano at Rome with a galilee. The galilee was considered less sacred than other portions of the church; hence its name.

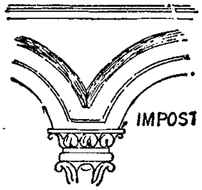


Fig. 2250.—Capital, showing Impost.

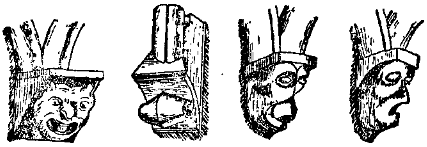


Fig. 2251. Fig. 2252. Fig. 2253. Fig. 2254.
Figs. 2251 to 2254.—Masks.

Gargoyles.—Examples of these curious forms of ornament are shown in Figs. 2241 to 2243.

Honeysuckle.—Figs. 2244 to 2249 show varieties of the honeysuckle ornament.

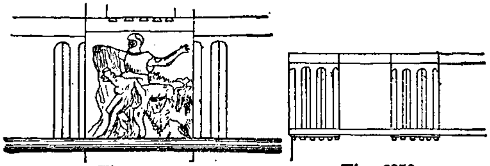


Fig. 2255. Fig. 2256.
Figs. 2255 and 2256.—Metopes.

Impost.—Impost (Fig. 2250) is the upper member of a capital, or the horizontal moulding at the top of the cap of a pier, pillar, or pilaster upon which an arch rests, and by which the carved work and mouldings of the fascia or spandril between the arches are supported. When there are no mouldings the impost is the block of stone immediately below the springing of a semicircular arch, or the block containing the skewback against which any other arch abuts.

√**Loggia.**—Loggia is a gallery open to the air in front, but forming a shelter from the weather, and may be used as a look-out.

Mask.—Mask is the name given to carved representations of faces, frequently grotesque (Figs. 2251 to 2254), used as finishings to Norman corbels, and as ornaments in string-courses, friezes, etc.

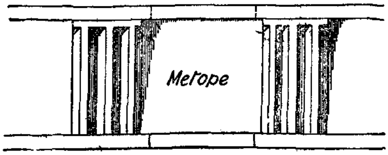


Fig. 2257.—Metope.

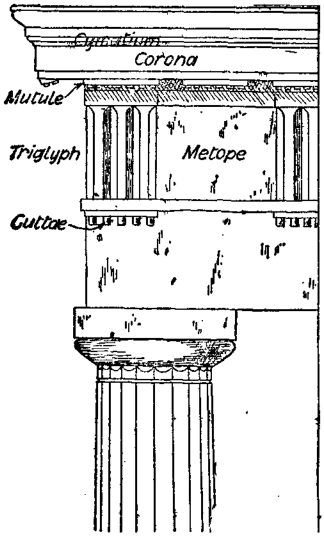


Fig. 2258.—Metope, Triglyph, etc.

Metope.—Metope is the square space between two triglyphs in the frieze of the Doric order; it is sometimes left plain, at other times decorated with sculpture, in low relief, of the human figure and animals (see Figs. 2255 to 2258).

√**Monopteral.**—Monopteral is a round temple with neither walls nor cella, but only a cupola or dome supported upon columns.

√**Naos.**—Naos is that part of a temple within the walls equivalent to the modern nave.

Narthex.—Narthex is a part of the early Christian church separated from the rest by a railing or screen, and to it the penitents were admitted. In some cases it was a vestibule or porch outside the church itself, extending along the whole breadth and covered by a lean-to roof against the church. In other cases it was in conjunction with an atrium or open courtyard containing a fountain at which persons could wash before going into the church.

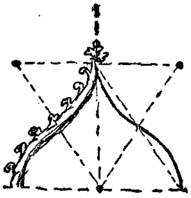


Fig. 2259.—Ogee Arch.

Ogee Arch.—An ogee arch of the Decorated period is illustrated by Fig. 2259, which shows the method of obtaining the centres for the four curves.

Parapets.—Battlemented, embattled, and crenellated mean the same thing. These terms imply the finishing of a parapet wall with alternate openings and projections, called merlons and embrasures (see Figs. 2260 to 2262). Castellated is finishing the top of a tower with an embattled parapet. Machicolated is projecting the parapet wall, embattled or otherwise, over a series of corbels, connected by small arches containing openings at the floor level of the rampart for the delivery of missiles on an enemy below (see Fig. 2263). Pierced refers to perforations, foliated or otherwise, in a parapet (see Fig. 2264).

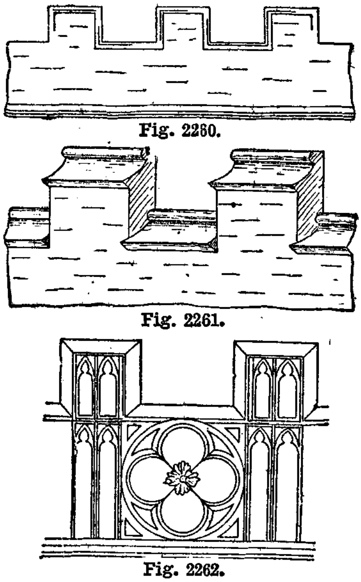


Fig. 2260.

Fig. 2261.

Fig. 2262.

Figs. 2260 to 2262.—Battlemented Parapets.

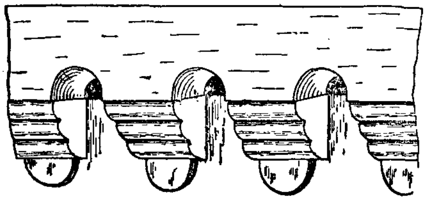


Fig. 2263.—Machicolated Parapet.

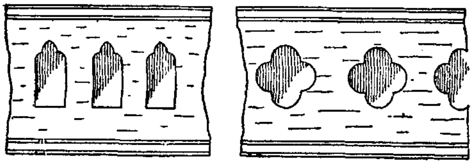


Fig. 2264.—Pierced Parapets.

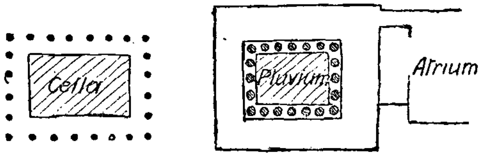


Fig. 2265. Fig. 2266.

Figs. 2265 and 2266.—Peristylium Arrangement of Columns.

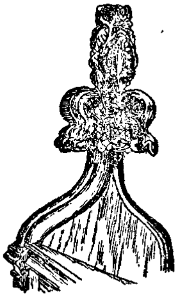


Fig. 2267.—Poppy-head Ornament.

Peristylium.—This term relates to the columns surrounding a temple, and sometimes in a court surrounding a tank (see Figs. 2265 and 2266).

Poppy-head.—The poppy-head ornament is shown by Fig. 2267.

Portcullis.—The portcullis (Fig. 2268) is an example of Tudor ornament.

Rood.—Rood is a large crucifix, or cross with figure of Christ, placed in Roman Catholic churches over the entrance to the choir or chancel, or upon or over the screen dividing the chancel from the nave (see Fig. 2269); hence the terms rood-screen, rood-loft, etc. Accompanying the crucifix, and on each side of it, are sometimes found figures of St. John and the Blessed Virgin Mary.

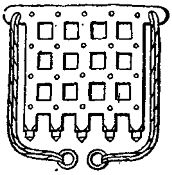


Fig. 2268.—Portcullis Ornament.

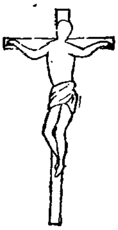


Fig. 2269.—Rood.

Rose.—The rose is, like the portcullis already illustrated, an example of Tudor ornament. It is shown by Fig. 2270.

Scoinson Arch.—Scoinson arch, or sconchon arch, is the name given to an arch thrown across the angles of a square tower to support the alternate sides of an octagonal spire (Fig. 2271). Similarly the diagonal pieces at the corners of a frame to stiffen it are called scoinson pieces.

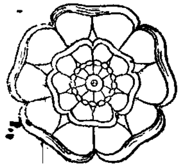


Fig. 2270.—Rose Ornament.

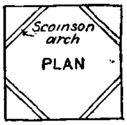


Fig. 2271.—Scoinson Arch.

Sgraffito.—Sgraffito or scratched work is a variety of pointing in which a light surface is scratched to expose a darker ground.

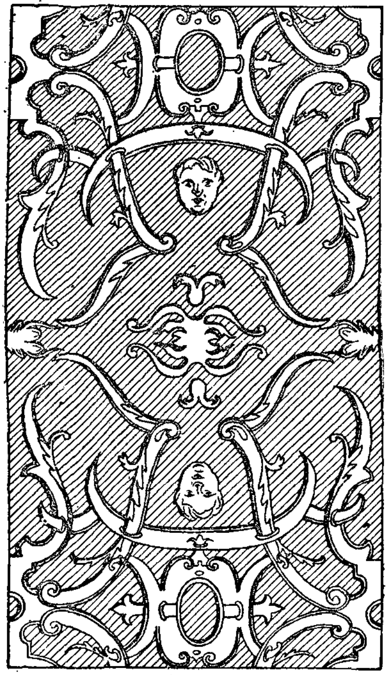


Fig. 2272.

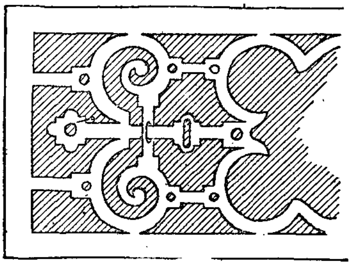


Fig. 2273.

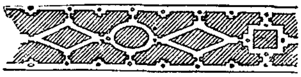


Fig. 2274.

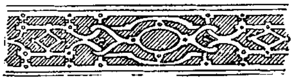


Fig. 2275.

Figs. 2272 to 2275.—Strapwork.

Strapwork.—Figs. 2272 to 2275 show various examples of strapwork in Jacobean architecture.

Tabernacle.—Tabernacle (Fig. 2276) in a *Roman Catholic* church is the movable structure, like a small model of a temple, in marble, metal, or wood, placed on or at the back of the altar, in which the consecrated elements or the Eucharist and the sacred vessels are kept. It is usually richly ornamented, and when of marble or metal has usually an inner shrine or receptacle of wood lined with silk. Tabernacle work is a term applied to carved canopy work with pinnacles, tracery, crockets, and other enrichments over a pulpit, sedilia, tomb, niche, or stall.

Telamones.—Telamones are figures of men used in the same manner as caryatides (see p. 512), which are female figures used to support an entablature instead of columns. They are sometimes called Atlantes, and also Persians.

Tenia, or Tape.—Tenia, or tape, is the name of the narrow band or fillet separating the architrave from the frieze in the Doric order.

Triforium.—The triforium (Fig. 2277) consists of a blind gallery or middle story under the aisle roof of a church, and separated from the nave by a series of openings or an arcade (Fig. 2278). Examples: Beverley, Westminster, etc.

Zigzag Ornament.—Some examples of the zigzag ornament already referred to are shown by Figs. 2279 to 2284.

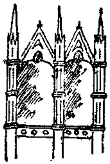


Fig. 2276.

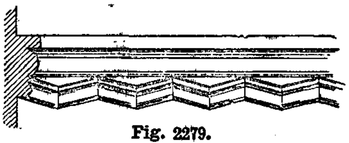


Fig. 2279.

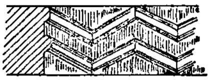


Fig. 2280.

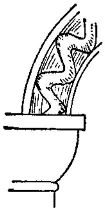


Fig. 2281.

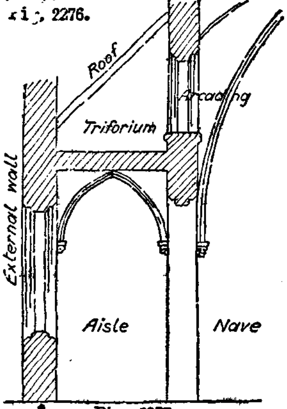


Fig. 2277.

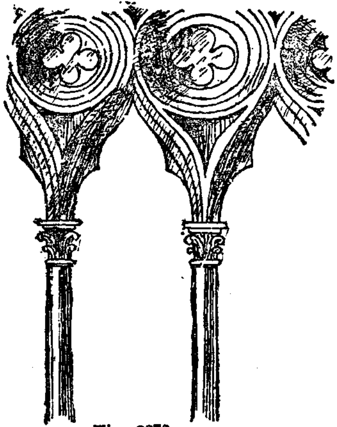


Fig. 2278.

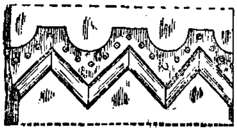


Fig. 2282.

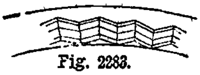


Fig. 2283.

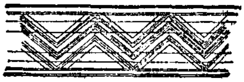


Fig. 2284.

Fig. 2276.—Tabernacle. Figs. 2277 and 2278.—Triforium. Figs. 2279 to 2284.—Zigzag Ornament.

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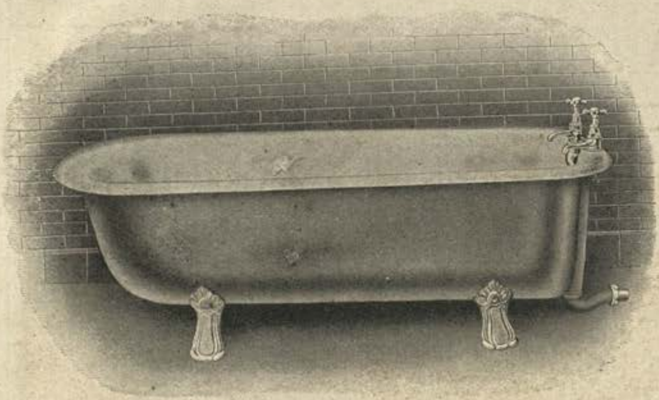

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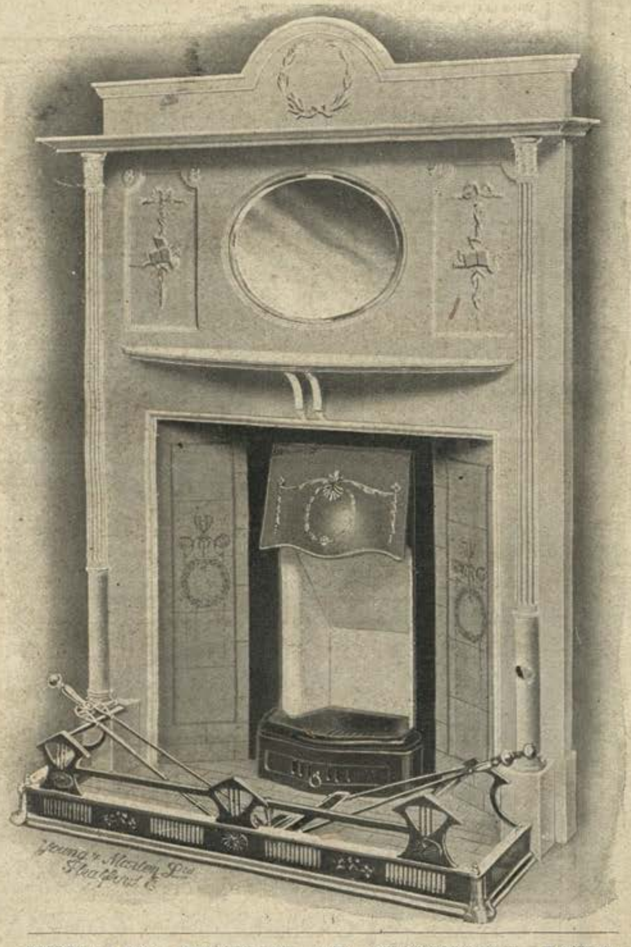

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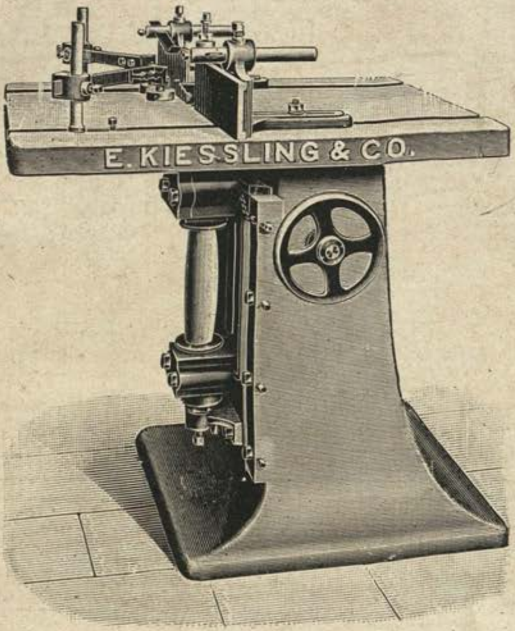
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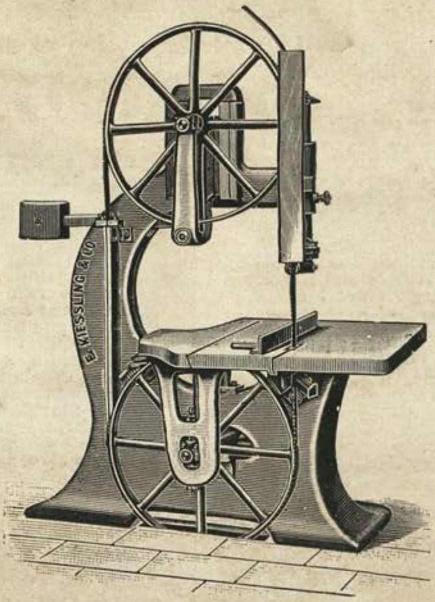
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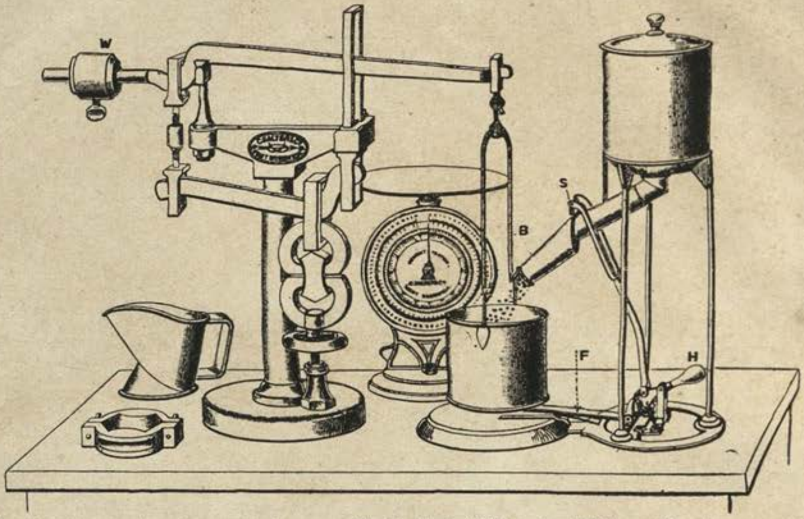

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